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Engineering and Contracting

Devoted to the Economics of Civil Engineering Design
and to Methods and Costs of Construction

VOLUME XLI

January-June, 1914

135003
29/10/14

THE MYRON C. CLARK PUBLISHING CO.
608 South Dearborn St., Chicago, Ill.

INDEX

VOL. XLI, JANUARY=JUNE 1914

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Engineering and Contracting

Devoted to the Economics of Civil Engineering Design
and to Methods and Cost of Construction

Volume XLI. CHICAGO, ILL., JANUARY 7, 1914. Number 1.

The Panama Canal.

The total bond issue authorized for constructing the Panama Canal was in round figures $375,000,000. The total appropriations made to July 1, 1913, and chargeable against this authorized bond issue were $338,800,000. On July 1, 1913, there remained of this sum appropriated as cash on hand a total of some $21,000,000. The expenditure for canal construction to July 1, 1913, had been therefore about $317,800,000. Including the cash on hand and the remainder of the authorized bond issue available for appropriation there was on July 1, 1913, some $57,200,000 of the original bond issue remaining to complete the canal. All of these sums are exclusive of appropriations for military defense of the Canal Zone and these had up to July 1, 1913, amounted to nearly $10,700,000. What will be the final cost of the canal? The 1906 estimate of the minority members of the International Board of Consulting Engineers was $139,705,200. The revised estimated of 1908 increased this amount to $297,766,000.

In this issue we publish an article describing the structures of the Panama Canal and the principal methods and costs of performing the construction work. From this article the engineer can see for what the expenditures so far made have been made, in what manner this money has been expended and, finally, what work remains to be paid for. The purpose of this article, however, is not to furnish a basis for speculation on the total cost to the United States of its trans-isthmian waterway; its purpose is to indicate in what manner a great piece of engineering construction has been planned and prosecuted by American engineers.

The plan of the Panama Canal is an American plan. European engineers who have had any part in trans-Panama canal planning have favored sea level canals. De Lesseps could conceive of no other than a sea level canal. It is true that he modified his original plan and permitted the substitution of a lock canal plan in place of the plan on which he began work, but he was driven by necessity to submit to the change. The new French canal company which took over the debris of De Lesseps' efforts was likewise constrained to accept the lock canal plan. When the International Board of Consulting Engineers, assembled by the United States in 1906 to decide on a canal plan, made its recommendations every European engineer of the Board was allied with the majority, which favored a sea level canal. The minority members, who favored a lock canal, were all American engineers. The Isthmian Canal Commission and its chief engineer, John F. Stevens, supported the minority recommendation of a lock canal and was in turn supported by the Secretary of War, Mr. Wm. H. Taft. There was a division of opinion in Congress so that the Senate might for a while "play politics," but the decision finally favored a lock canal. The lock canal plan was distinctly the American plan for a Panama Canal.

The lock canal plan was not only distinctively American, but the particular plan for a lock canal which was decided upon was distinctive from all previous plans for a lock canal. All other plans had sought for the longest possible stretches of sea level channel; their locks were placed the farthest possible distances inland. Traditional conception of the immeasurable superiority of ocean-level waterway, one might imagine, so overshadowed trans-isthmian canal planning that thought of a lock was to be dared only when no possible way remained for a channel open to the sea. The five American engineers of the minority of the International Board of Consulting Engineers reversed all precedent: instead of seeking the longest possible sea levels they purposely stopped both Atlantic and Pacific sea level channels at the first points inshore where it was practicable to build locks and secure super-levels of economic length. The first Atlantic locks were located at Gatun only a few miles from the shore of Limon Bay; the Pacific locks were located at La Boca only a little beyond the shore line of the Gulf of Panama. Unstable foundations at La Boca caused the original location of the first Pacific locks to be changed, but the Gatun location remained and Gatun lake and the co-ordinate dam and locks of Gatun constitute the unique feature of the American Panama canal plan.

Gatun lake is the summit level of the Panama Canal and its waters carry ships for 32 miles of the total 50 miles of Panama channel from deep water to deep water in the opposite oceans. The structural and the constructional features of Lake Gatun are explained elsewhere; its enormous significance as a structural part of the Panama Canal can hardly be overestimated. Credit for making Gatun Lake a feature of the American Panama canal plan belongs first to John F. Stevens, who realized its possibilities and inaugurated investigations in proof of its constructional practicability and then, when the International Board of Consulting Engineers took up its work, contended fiercely for a lock canal with Gatun Lake as its summit level. Credit is secondarily due to Engineers Ripley, Randolph, Stearns, Noble and Abbot, who, as the minority of the International Board of Consulting Engineers, supported Mr. Stevens and the Isthmian Canal Commission and lent their influence and high authority in upholding the plan before Congress.

Gatun dam, next to Culebra Cut, from the first held most firmly of all the Panama Canal structures the notice of the American people. It was of all the canal structures subjected most to criticism. In a way this fact is curious. The La Boca dam and lock foundations disclosed from the first conditions alarming to engineers yet work nroceeded for some time on these Pacific counterparts to the Gatun locks and dam and no American newspaper gave more than a passing news dispatch to them, while alongside they were printing most alarming articles on the Gatun dam whose foundations had from the first been held unquestionably stable by all the engineers of the canal. Again, too, the engineering judgment and foresight which approved the Gatun dam were assailed, while the one great engineering miscalculation at Panama, the rock slides at Culebra, drew scarcely a single criticism.

Culebra Cut was undoubtedly in the popular mind the main spectacular feature of the canal at Panama. To engineers it presented at first no consideration beyond those of an ordinary railway cutting in the United States except the single consideration of its enormous dimensions The French engineers who began the cut held it to be merely the problem of removing a calculated prism of earth and rock. The French company's geologists, two famous authorities in their science, could see no danger in possible slides and Cucaracha slide had already shown evidence of its power. The first Isthmian Canal Commission of 1901, officially reported, as is quoted elsewhere, that slides of rock were a remote danger. Advocates of the sea level canal plan when the question of the canal plan was before the Senate in 1906, were strenuous in their contention that slides were matters of unimportance in planning a cut of whatever dimensions at Culebra. Blame for these opinions, fatuous as they were, cannot be severe nor can it be placed on any one expert more than another for the opinion was general that Culebra cut offered no obstacles to construction beyond the immense volume of material to be removed from the calculated prism. Even after rock slides developed at Culebra and began to multiply confidence was felt that each was the last.

It has been noted in detail elsewhere what volumes of excavation were added to Culebra Cut by slides, but a summary in other terms is not out of place. According to the estimates of the majority and the minority of the International Board of Consulting Engineers a sea level canal would require a cut at Culebra of 110,000,000 cu. yds., and an 85-ft. level lock canal would require a cut at the same point of 53,800,000 cu. yds. The estimate of 1908 called for a cut of 78,000,000 cu. yds., of which 13,000,000 cu. yds. were due to prism enlargement. In 1910 the estimate was increased to 84,000,000 cu. yds., the addition being to provide for slides already developed. In 1911, the estimate was made 89,000,000 cu. yds., the increase being due to slides. Again, for the same cause, in 1912 a new estimate of 94,000,000 cu. yds. was made, and on June 30, of the year just passed an increase to over 100,000,000 cu. yds. was estimated. These figures tell the story of engineering miscalculation at Culebra. The questions which occur are: What would have been the volume of slides had a sea-level canal, with a cut some 80 ft. deeper at Culebra, been adopted for construction? Would sea-level canal advocates have remained advocates of a sea-level canal had they anticipated any considerable portion of the trouble which slides have caused? Indeed, would not the lock canal advocates have considered seriously a higher summit level and more locks had they foreseen what volumes of sliding rock would have to be excavated at Culebra?

There is little profit in speculating on what might have happened had the true conditions of rock stability at Culebra been anticipated. It is better to notice that, because of constant increase in excavating efficiency, costs were so considerably reduced below the originally estimated cost that the total cost of Culebra Cut with its 20,000,000 extra cu. yds. of excavation will probably not materially exceed the estimated total. This fact is one of the marks of credit which stand on the records of the engineers who have directed the work at Panama. Another greater mark is that all the millions expended have not included the smallest of sums misappropriated.

There have been many of the opinion that day labor construction of the Panama Canal was a mistaken policy. There is none, we feel, who believes that any engineer having direction of day labor work at Panama did not utilize every cent honestly and with every effort to secure the greatest return possible from its expenditure. Under these conditions day labor construction reaches the greatest economy that it is capable of attaining. Under no other conditions and seldom under these conditions can day labor construction compare in economy with contract construction.

The enormous output of excavated material at Culebra has been one of the most noted items of the canal work. It was the fashion during the earlier days of the work to contrast this output with the monthly or yearly excavation records of the French engineers. There was more of vanity than of justice in these comparisons. The difference between French yardages and American yardages at Culebra and indeed everywhere on the canal, indicates the increase in efficiency of excavating machinery between 1885 and 1905. The notable thing is not that American yardages so much exceeded French yardages, but that the French directors of canal work secured as great outputs as they did with the plant with which they had to work. There was another advantage possessed by the American builders which their French predecessors did not have. This was immeasurably more healthful living and working conditions. In this matter also the French engineers have been somewhat over-blamed. They did not quite likely lay the stress that they might on sanitation, but they did not in 1882-5 know sanitation as it was known 20 years later and neither they nor anyone knew that mosquitoes and no other agent propagated yellow fever and its almost as objectionable if less deadly companion disease, malarial fever. Mosquito extermination counted far more toward the faster work at Panama of Americans as compared with Frenchmen than did so-called superior Yankee push and efficiency.

The work of sanitation at Panama does not require much space to describe but it was the most nearly marvelous of all the work of the Americans at Panama. At no previous time could it have been accomplished so effectively, because until 1900 it was not known that yellow fever and malaria were misquito-borne diseases and then or even four years later when Americans occupied Panama, the fact was the accepted belief of only a few. It has been said that sanitation at Panama was merely a matter of intelligent administration and this is true but the intelligence required was of the kind that could take every advantage of scientific research and leave no precaution untested because of previous beliefs.

The canal at Panama is above all the work of the engineer as a builder and as a sanitarian but it is also the work of the engineer as a designer, and the designing engineer deserves co-ordinate mention with the construction engineer and the sanitary engineer. The locks with their gates and valves and safety devices, the power plants and the system of power transmission and control called for designing ability of high character. All general plans were worked out by the canal engineers and then planned in detail in conjunction with the engineers of the manufacturers who built the several parts and appliances. A glance at the list of the manufacturers which is given elsewhere and at the money value represented by their contracts will demonstrate more forcefully than many words the character and magnitude of the designing engineers' work at Panama.

This discussion of the work at Panama though confined solely to general features has touched only the main points deserving notice. It has, however, reached the limits of available time and space and the reader must supply from the article which is printed elsewhere in this issue, the many deficiencies that exist.

A Decrease in the Use of Mineral Waters.

During 1912 1,500,000 gals. less mineral and table water were consumed in the United States than during the year 1911. This decrease in consumption represented a total decrease in the expenditure for such waters of $250,000 for the year. A bulletin recently issued by the United States Geological Survey states that 62,000,000 gals. of mineral water were consumed in 1912, and cost, at the source of supply, about $6,500,000. This figure does not, of course, represent the full cost to the consumer. The price of mineral waters ranges from a minimum for the table waters sold in large containers to a maximum for waters possessing medicinal properties and for the higher grade of table waters which are bottled and carbonated. The decrease in consumption of the waters under consideration is a matter of interest to engineers who are engaged in water supply practice.

While perhaps not the sole cause, we feel sure that first among the causes leading to the reduction in the use of mineral water during 1912 is the greatly improved quality of many public water supplies. The installation of numerous emergency hypochlorite plants in connection with domestic water supplies has given that portion of the public most likely to buy bottled water for table use greater confidence in the purity of the public supply. The rapid advance in the adoption of filtration for surface supplies has of course also had its influence in decreasing the use of table waters. Readers will recall that the chief reason that work on the construction of the Minneapolis filter plant was carried on during the winter was to have the plant ready for operation as soon as possible so as to save the people of that city the $40,000 per month which they had been paying for bottled water for table use.

The saving indicated for the year 1912 is, of course, comparatively small and is of interest chiefly as indicating a growing public confidence in the purity of public water supplies. With the general adoption of filtration for surface supplies and the universal availability of emergency disinfecting plants, the expenditure of money for bottled water for table use will become practically *nil*.

Water Works

A Discussion on the Construction and Operation of the Several Types of Meters and Their Adaptability to Various Kinds of Service-Performance of Water Meters in St. Louis.

As early as 1880 the St. Louis water department had a few water meters in service. The only meter then known was the Worthington plunger, which is very similar to a double piston pump. A so-called modern type of water meter such as the rotary piston began to displace the plunger type about 1892 and has, till recently, continued to be the type preferred by the department. The disk and velocity types followed later, but the velocity type was considered too inaccurate on small flows and the disk type [illegible] unreliable. The water department has changed its policy and is now giving preference to the [illegible] and velocity type over the more expensive rotary.

In a [illegible] paper before the Engineers' Club of St. Louis [illegible] Engineer in Charge of the Distribution System, [illegible] the [illegible] of the [illegible] meters which have been used by the St. Louis water department [illegible] their performance [illegible] St. Louis. The [illegible] here given [illegible] American [illegible]

It is essential to know the construction and operation of the various types in order to compare their performance and discuss their adaptability to various kinds of service. The [illegible] type, the duplex plunger, is mentioned only as a matter of historic interest for it is [illegible] for measuring water supplies. It should be used though, to measure hot water, oil and other liquids which would ruin the hard rubber parts used in all modern types. It will suffice to describe its operation as similar to that of a duplex water pump with its parallel cylinders and sliding valve. A driving arm and pawl transmit the reciprocating motion of the plunger to a ratchet wheel on the under side of the recording dial. The other types can be classed as modern and are of the same general scheme; they differ principally in their measuring chamber and are classified accordingly. The water passing through the measuring chamber operates a hard hubber piston, disk or wheel. Within the same casing, lubricated by the circulation of water, is a train of bronze gears which reduces and transmits the piston motion through a stuffing-box to the recording dial. The dial and gear train spindles are connected by a pair of gears which can be changed so as to correct any small inaccuracies of registration. The modern types may then be classified as rotary, disk and velocity. There are several models on the market which are a combination of these. The Venturi meter is not considered in this paper, for it is not used on ordinary city service connections.

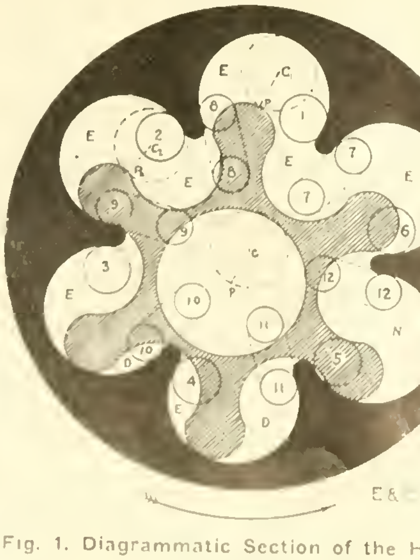
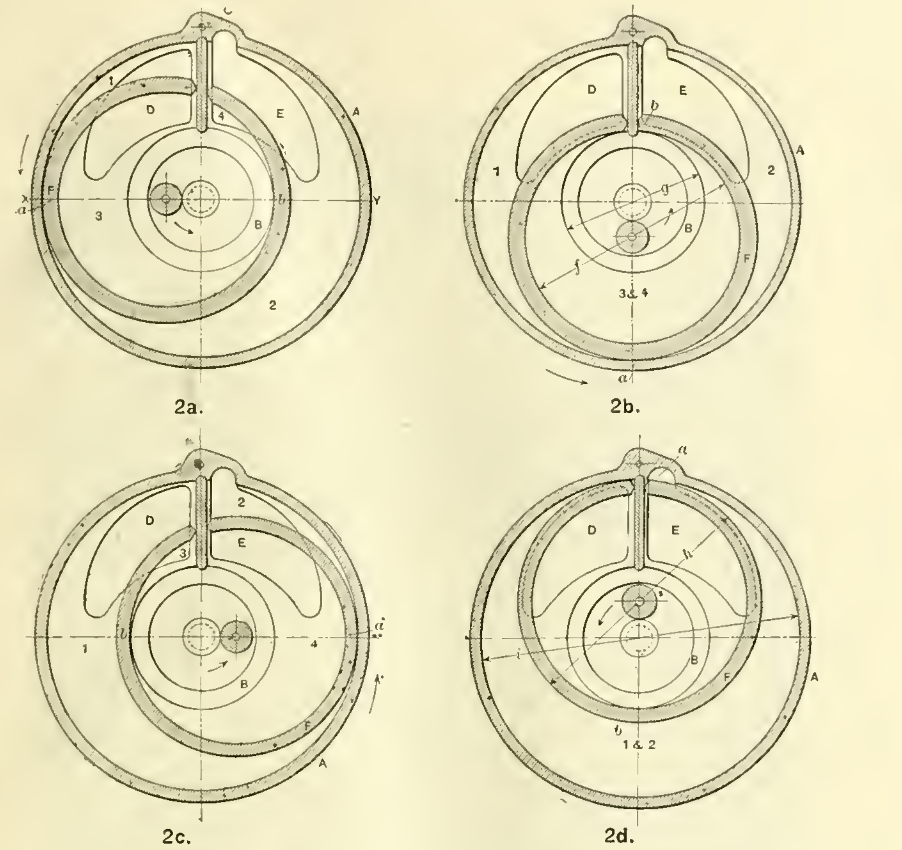


Fig. 1. Diagrammatic Section of the Hersey Rotary Water Meter.

The Crown was the first of the rotary piston type, and was introduced in St. Louis early in the nineties. The Crown was soon followed by another rotary, known as the Hersey Rotary. The general arrangement of the parts, piston and chamber walls bears such similarities and contrasts to the Crown meter that one might easily imagine the Crown piston and chamber adapted to a circular motion instead of a series of swinging motions. The water enters the measuring chamber through the ports in the bottom, and leaves through the opening at the center of the top.

The hard rubber ring and piston are hollow, and the piston is of such specific gravity that it weighs in water about 35 per cent of its weight in air. Figure 1 is a horizontal section through the measuring chamber, showing the position of the piston with respect to the inlet and outlet ports in the bottom. At the instant shown, water enters the chambers *E* through the inlet ports 1, 2, 3, 4 and 6 in the bottom. It will be noticed that the six inlet ports are twice as large in area as the outlet ports 7, 8, 9, 10, 11 and 12, because there is a corresponding set of outlet ports in the top plate. At the same instant water is discharging from chambers *D* through the outlet duct ports 10 and 11 to the cylindrical opening in the center of the piston, which in turn is always open to the outlet of the meter. The inlet port 5 and outlet port 12 are on the point of opening. In a complete revolution every point in the piston describes a circle of radius *PC*. In its motion it is quite evident that all surfaces of contact are sliding, in contrast to that of the Crown, where the sides are in rolling contact while the top and bottom only are in sliding contact. The increased wear of all sliding surfaces in the Hersey Rotary is very noticeable, and the friction caused by the sliding contact of these roughly worn surfaces seem to account for its marked loss in sensitiveness. The volume discharged per revolution is the volume of the chamber minus that displaced by the piston and the cylindrical outlet pocket in its center.



2a.

2b.

2c.

2d.

Fig. 2. Sectional Views of Empire Rotary Water Meter, Illustrating Various Positions of Ring-Shaped Piston.

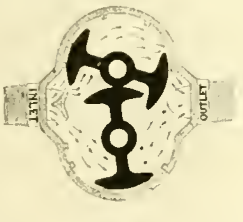


Fig. 3. Sectional Diagram of Union Rotary Meter.

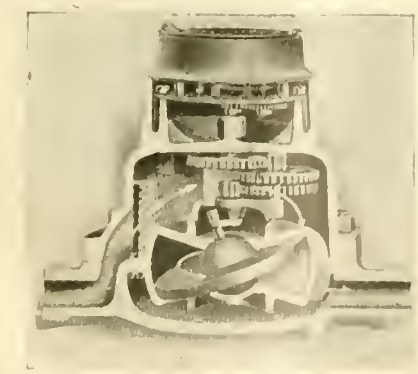


Fig. 4. Disk Meter With Flat Disk and Conical Chamber.

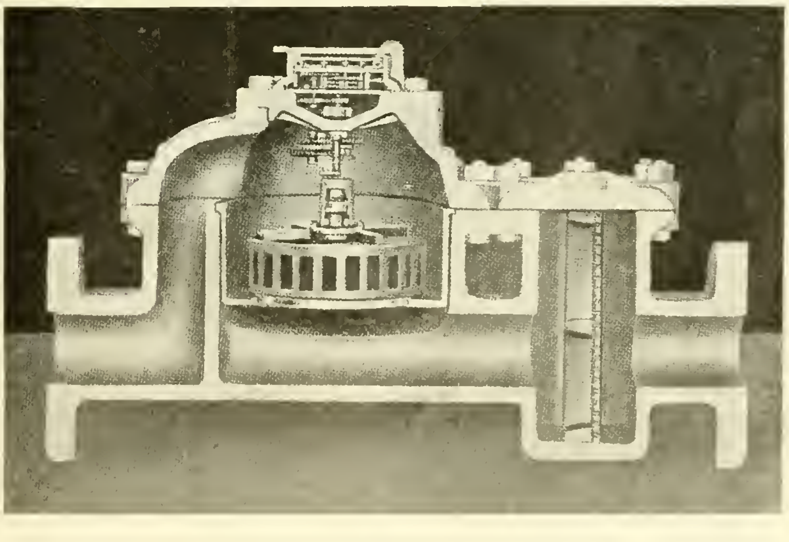


Fig. 6. Velocity Type of Meter, in Which Water Enters at Center of Wheel and Discharges at Outer Edge.

Another meter classed as rotary is the Empire. The motion of the ring-shaped piston might be described as oscillating. The center of the piston, Figs. 2a-2d, describes a circle about the center of the chamber, while the slot in the ring reciprocates along the stationary diaphragm. The circular motion is controlled by a hard rubber pin in the center of the piston, sliding in a circular ring *B* concentric with the chamber. In Figs. 2a-2d the water enters port *D* in the bottom plate and discharges from *E* in the top plate.

The piston is so controlled by the diaphragm *C* and ring *B* that there is always some point *a* on the outer edge of the piston in contact with the outer chamber wall, while diametrically opposite on the inside of the piston ring there is continually a point *b* in contact with the inner wall, the guide ring *B*. Consequently this diametrical line of contact *ab* separates the inlet from the outlet spaces. As the pressure acts in a direction against the projection of this diametrical line, it acts parallel with the contact surfaces, and thus excessive friction is avoided. In Fig. 2a the spaces 1 and 3 are filling while 2 and 4 are discharging, thereby causing the piston to move in a direction indicated by the arrow. On moving to position in Fig. 2b the inlet space 1 has enlarged; 3 and 4 have combined and are about to discharge through port *E*; while the outlet space 2 has diminished. Fig. 2c shows the inlet space 1 still enlarging, and at the same time the newly formed inlet space 3 and the outlet spaces 4 and 2 as diminishing. In Fig. 2d the inlet space 1 has become a maximum and is about to discharge and become 2 of Fig. 2a. This would bring the piston into its original position of Fig. 2a. From the foregoing explanation it is quite evident that the annular space between the outer and inner walls *A* and *B* of the chamber is filled and discharged in each oscillation of the piston.

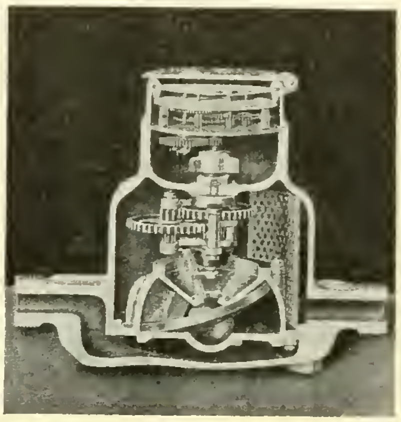


Fig. 5. Disk Meter With Conical Disk and Flat Bottomed Chamber.

The Union Rotary Meter, as shown in Fig. 3, is another of the rotary type. The design of the rotors is similar to that of a well-known type of blower. The rotors are made of hard rubber and are fixed in relative position by two elliptical hard-rubber gears. The volume per revolution will be equal to the volume of the chamber minus the volume displaced by the two rotors. The direction in which the rotors turn is controlled by a V-shaped guide vane at the inlet which divides and deflects the incoming stream so that it flows around by the wall of the chamber to the outlet.

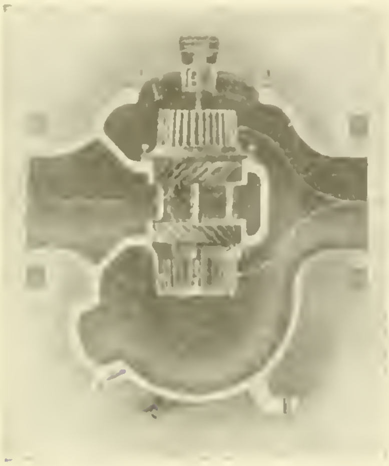


Fig. 7. Velocity Meter With Two Reaction Wheels on Same Shaft, Giving Balanced End Thrust.

The most popular meter in sizes smaller than 3 ins. is the disk type (Figs. 4 and 5), so-called from the shape of its hard-rubber piston. The disk-shaped piston may be said to mutate or wobble about the ball and socket bearing at its center. The chamber in which the disk mutates may be described as a portion of a sphere enclosed by the zone generated by the revolution of an arc about the diameter as an axis. There are two shapes of disk, one flat and the other conical, requiring a corresponding difference in the chambers. Figure 4 shows a meter with a flat disk; the top and bottom of the chamber are conical in shape and symmetrical. Figure 5 shows a meter where the disk is conical and the bottom of the chamber flat. A diaphragm extending from the outer wall of the chamber to the ball of the disk at the center divides the chamber so that the water entering at one side of the diaphragm travels around the annular-shaped chamber, forcing the disk before it, and is discharged from the opening at the other side of the diaphragm. The two lines of contact between the disk and top and bottom of the chamber divide it into four parts, of which two are continually filling and the other two discharging. This results in a continuous flow, and the volume displaced per revolution is the volume of the chamber minus that displaced by the disk.

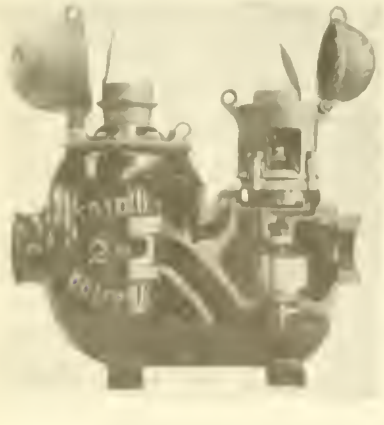


Fig. 8. Compound Meter Designed to Give Minimum Impedance to Large Flows and Accurate Measurement of Small Flows.

The foregoing types measure water by positive piston displacement, in contrast to the velocity type, which measures the volume of flow by its velocity. These velocity types operate as reaction water wheels. Figure 6 shows a velocity type meter where the water enters at the center of the wheel, reacts against the curved vanes and discharges at the outer edge. Another design of velocity meter is shown in Fig. 7, where two reaction wheels are mounted on the one shaft so that the end thrust is balanced by the water reacting against the blades from both ends of the runner. The weight of the runner in water is carried by agate or hard-rubber bearings at the bottom of the shaft. From tests made on the various types of velocity meters the double balanced wheel appears to be the most sensitive to low velocities.

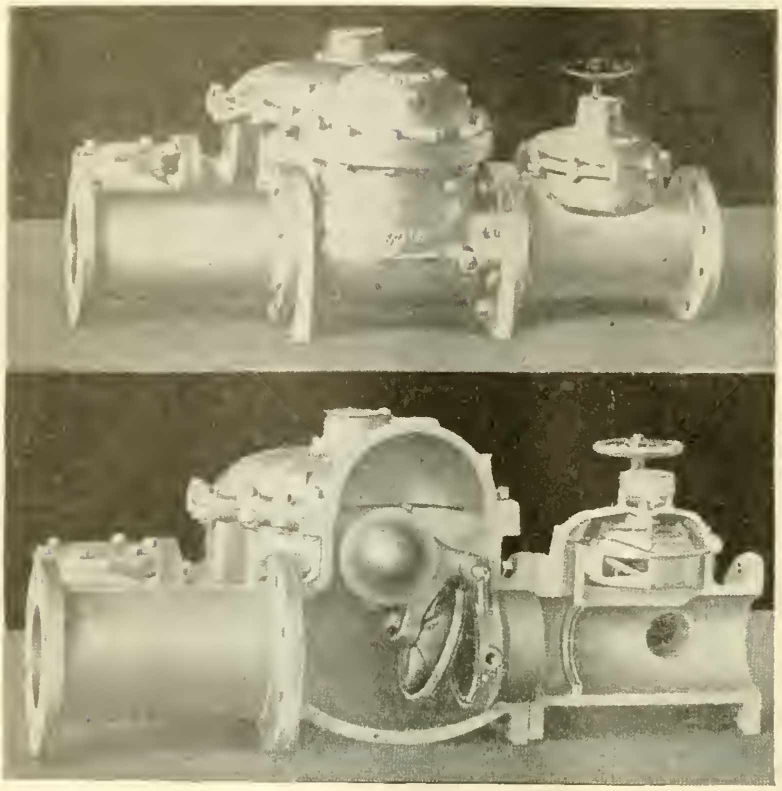


Fig. 9. Hersey Detector Meter—Suitable for Use on Fire Protection Services.

Figure 8 shows a compound meter which is designed to give a minimum impedance on large flows and to measure accurately small flows. On small flows the disk meter operates, and when its capacity is reached the weighted valve is unseated, closing the small meter and opening the velocity meter to operate on the larger rates of flow.

Meter manufacturers in the past have been endeavoring to construct a meter suitable for fire-protection service, and up to the present time only one make has been approved by the fire underwriters. The value of such a meter for detecting the theft of water from fire protection services will be understood by any one familiar with water works. A striking example of its use was made in a city of Michigan, where after installing one of these meters on the fire service of a manufacturer, it was found that water was being stolen at the rate of about $5,000 a year. From the evidence it appeared that this theft had been going on for about four years; the jury rendered a verdict against the defendant for $15,812.76. Figure 9 shows such a meter. The main pipe is equipped with a weighted check valve in front of which is a baffle ring to deflect about 5 per cent of the main-pipe flow through the proportional meter. A metered by-pass half the diameter of the main line connects the two sides of the check valve. An annular groove in the seat of the check valve is opened to the atmosphere when the valve is seated. This arrangement holds the valve closed under a pressure equal to the difference between the total resultant hydraulic pressure on both sides of the valve gate and the total atmospheric pressure on the groove of the valve seat. The groove in the seat is of such area that the check valve is unseated when the pressure drop around the valve is 6 per cent. The metered by-pass registers the entire flow at rates which are not sufficient to open the valve gate. On flows that keep the check valve open the proportional meter operates so as to register that portion which passes through the main line. The function of the ball weight is to reseat the check valve, for it would tend to float, as no excess pressure is created above it when unseated. Figure

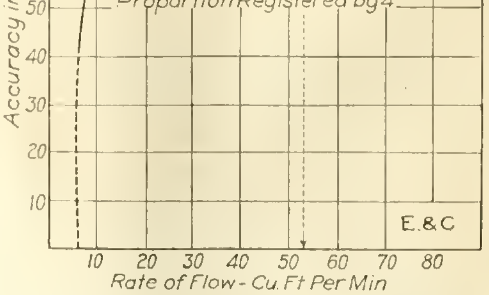


Fig. 10. Proportional Flow and Accuracy of Registration on Main Line and By-Pass at Various Rates of Flow Through a 4x12-in. Detector Meter.

10 serves to illustrate the proportional amounts and accuracy of registration on both the main line and by-pass at various rates of flow through a 4x2-in. detector meter. This detector meter is quite adaptable to services where both large and small rates of flow are desired. The feature in the detector of measuring large flows in proportion is used in a so-called "proportional meter." One model is a straight section of pipe with a baffle ring to deflect a portion of the water through a small meter, calibrated to register directly the total flow. The other model operates in the same way, but in addition is equipped with a by-pass, so that the meter can be repaired without interrupting the water service.

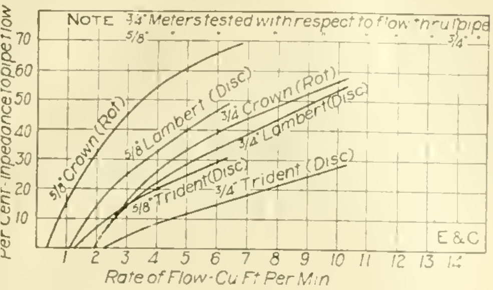


Fig. 11. Comparative Impedance to Flow of Various Types and Sizes of Water Meters.

The remainder of this paper will be devoted to the comparative performances and characteristics of the various types.

The curves of Figs. 11 and 12 serve to show the comparative impedance of the different types and sizes at various rates of flow. From these curves it is evident that near full capacity the impedance of the velocity type is much less than that of the disk type, which in turn is less than that of the rotary. On their smallest flows the impedance of any type is neglible. The fact must not be overlooked that the full capacity of the disk type can, within limits, be easily regulated by the size of inlet and outlet passages to the measuring chamber. The meter manufacturers are quite uniform in the matter of full capacity, disk mutations per cubic foot, and the size of measuring chamber. This uniform practice of piston speed represents the maximum that is consistent with a good wearing efficiency.

It has been the experience of the St. Louis Water Department that rotary meters lose their accuracy and sensitiveness gradually. They will still continue to operate when they have become so worn as to register as low as 60 to 80 per cent of accuracy. These large inaccuracies can be attributed to the wearing of the comparatively large sliding surfaces exposed to the grinding action of grit in the water. This excessive wear results in a large amount of play between the piston and the chamber, permitting water to slip through without acting on the piston. The sliding surface exposed to grit is far greater in the rotary type than in any other.

Our observations indicate that the disk type maintains its accuracy better than the rotary. The disk type generally maintains an accuracy of 90 per cent or better until it fails entirely. The maintained accuracy can be explained by the fact that the wearing surface of a disk is concentrated at the ball and socket bearing. This affords only a small surface exposed to grit and slippage. When the wear in the bearing allows sufficient play, the edge of the disk will bind in the chamber and so cease to operate and probably break. These disks are cheap to replace as compared to the pistons of the rotary type.

The velocity type, due to the small wearing surface exposed to grit, maintains its accuracy very well. It is used only in the larger size—that is, 3-in. and above, and is by far the cheapest in first cost and maintenance. It is also most accessible for repairs and is especially adapted to the services of consumers where large rates of flow are used, or where water is used only at long intervals of time. It is not particularly adapted to services that have small fixtures and openings, for the velocity type will not register leaks and will only partly register the small streams.

The disk type is about one-third less in first cost than the rotary, due in a measure to the greater amount of hard rubber and machinery necessary in the piston and chamber of the rotary type.

The disk type in the smaller sizes is more accessible to repair. One make embodies in the design properties for incurring a minimum amount of damage in case of freezing. The bell shape of the upper casing causes the stress of freezing in a vertical direction. The bottom is so designed that it will break before the other parts are permanently distorted. When the bottom is released the measuring chamber is so designed as to be free to fall apart in case it is subjected to an internal stress. Our experience finds that on replacing a broken bottom of such frozen meters the accuracy has not been affected. And so the measuring chambers of many makes of disk meters have special features which are advanced as arguments in their favor. The wearing surfaces of the measuring chamber of a disk meter are confined to the ball and socket bearing and the diaphragm. To reduce the cost of repairing a badly worn ball-socket bearing, some firms construct the disk chamber in three parts instead of two, so that only the top and bottom plates need be replaced when the bearing is worn. Some also construct the ball of the disk so that it can be taken apart and adjusted for wear by the insertion of paper between the halves. Excessive wear on the diaphragm caused by the tendency of the disk to rotate about its spindle must be avoided if the meter is to be reasonably accurate on small rates of flow. Several firms entirely avoid this wear on the diaphragm by a thrust roller which is placed in the edge of the disk at a point diametrically opposite the diaphragm and so takes the side thrust while oscillating

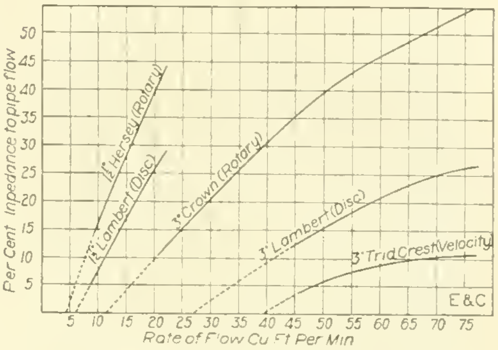


Fig. 12. Comparative Impedance to Flow of Various Types and Sizes of Water Meters

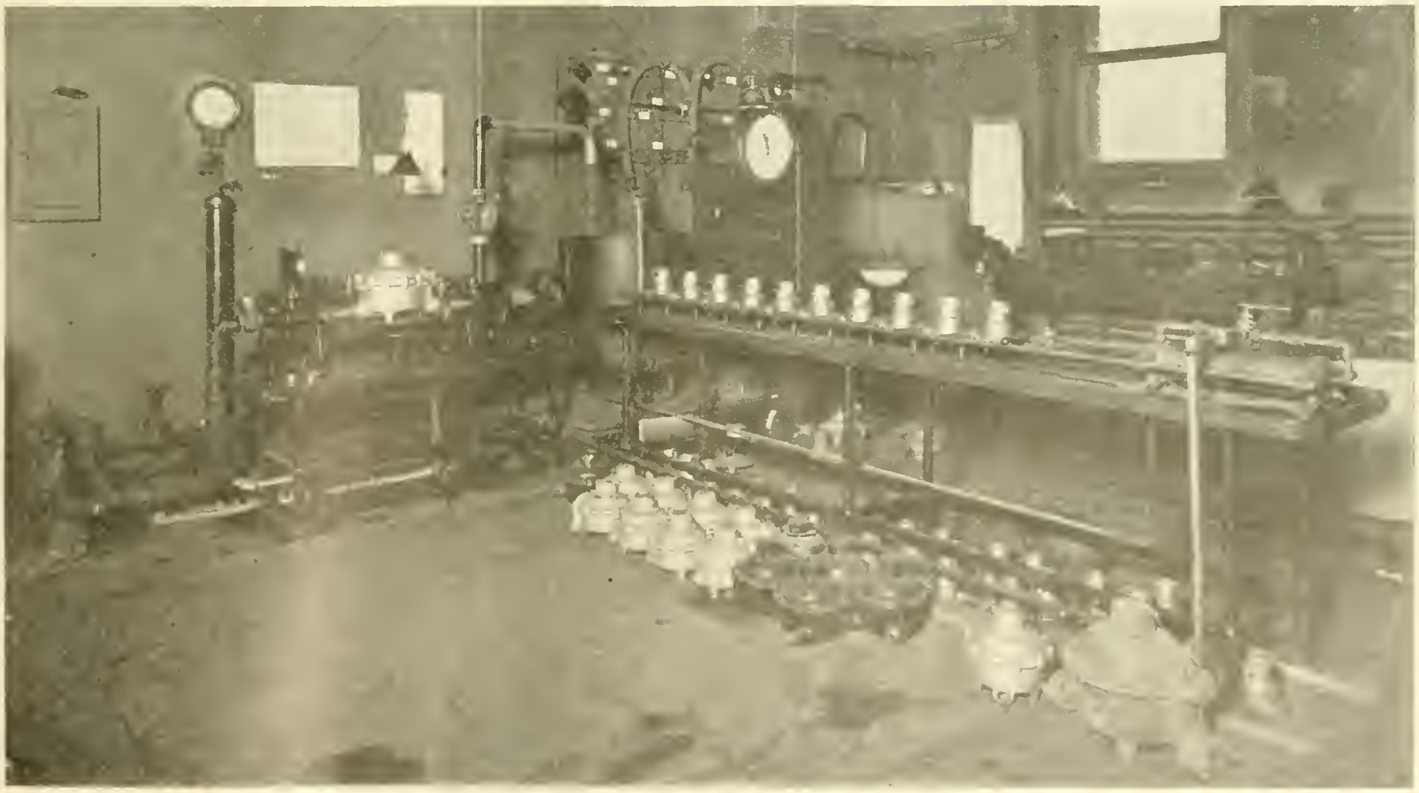


Fig. 13. View of Machine for Testing Small Meters, Ten at a Time.

in a vertical slot in the side of the measuring chamber. From the data which the department now keeps it will be able in a few years to determine whether or not this feature is an advantage. As previously shown, some meters have flat disks and others conical. The advantage claimed for the conical-shaped disks is that any section is a curve, so that it will embody the strengthening effect of an arch. Many firms strengthen the flat disks by a steel reinforcement. In the case of the flat disk the horizontal thrust of the water in motion is less than in that of a conical disk and is proportional to the sine of the angle between the disk and the horizontal.

The gear train in any meter which transfers the piston motion to the dial is generally made of phosphor bronze pinions, gears and shafts, while the frame is bronze. One difficulty experienced in the past with gear trains in St. Louis water is a heavy white deposit of calcium carbonate which becomes so thick as to interfere with their operation. This calcium deposit also closed strainers and passages and covered pistons and chambers with a hard white deposit. It is almost impossible entirely to remove this coating by a mechanical process. The department now removes this coating quickly and cheaply with a dilute solution of muriatic acid and so restores to service many meters and parts which were formerly condemned for scrap. The pistons of rotary meters being hollow and having small openings for the admission of water resulted in a deposit of calcium carbonate on the inside of such pistons. For example, a piston taken from service weighing 10½ lbs., after treatment in acid lost 2½ lbs. In all similar cases, the piston becomes more buoyant and responds to small rates of flow, whereas it failed to do so before the acid bath. Another difficulty experienced is in the gear train, and is due to the soluble salts in our water which set up an electrolytic action between the parts having different zinc contents. That is, the parts containing more zinc when exposed to the action of the electrolytic solution become electro-positive with respect to the parts containing less zinc, causing the parts high in zinc to disintegrate and leave the copper constituent in a porous and brittle form. This electrolytic action was recognized by Mr. Monfort, chief chemist of the St. Louis water department, and described by him in a paper before the American Water Works Association convention at New Orleans in 1910. As a remedy he suggested that the various parts should as nearly as possible be of the same alloy. Accordingly a change was made by one firm in the construction of the frame parts of the gear train so as to conform with Mr. Monfort's suggestion. The posts were combined with the upper part of the frame to make a unit and so lessen the parts that might otherwise be of a slightly different alloy. A clause containing the feature of a uniform alloy throughout the gear train has been embodied in a specification under which the city purchases meters. To prevent corrosion and increase the wearing efficiency, one firm constructs a gear train of hard rubber gears, hard-rubber thrust bearings and phosphor bronze pinions. Some firms use a gear train in which jewels serve as thrust bearings.

Seemingly the case of a meter becoming fast comes to our attention and such cases are so rare among the modern types that this was the only one seen in several years past. Upon examination of the measuring chamber of this meter it was found the meter had so worn [illegible] as to form [illegible] of the [illegible] plate, so that the [illegible] the length of the [illegible] per revolution [illegible] and the meter [illegible] at 90 per cent. [illegible] had been 110 per cent.

Two of the types [illegible] in general use. The [illegible] is a round dial, where [illegible] em-ploys hard-rubber bushings for the bearings. The other is known as the straight-reading type register, and consists of a set of rollers, one for each digit. The roller of the lowest digit is geared directly to the source of motion, while the others are operated by trippers on a countershaft. The vital point of a straight-reading register lies in the design of this countershaft and its trippers, because the trippers for the higher digits operate so seldom that unless some means is provided for preventing it, they become tightly corroded to the shaft, and in so doing the tripper shaft breaks when the trippers of the higher digits are brought into operation. There are two designs which are covered by patents, one providing for an intermittent motion of the countershaft and the other for a continuous motion. In the former, the first tripper is fastened to its shaft so that the shaft turns once for each revolution of the first digit roller. In the case of the latter, the tripper shaft is geared directly to the main spindle, so that it operates continuously with the source of motion. To prevent tampering with meters, we deem it necessary to seal the dials only. Tampering with the measuring chamber or train gear is hardly ever done, and is so easy to detect that the department considers it over-precautious to seal the main casing, especially in view of the fact that the meters are read once a month. The department now uses an aluminum seal which is exceedingly cheap.

Meters are purchased under specifications which are designed to admit of standard makes that will insure satisfactory performance as to wear, corrosion and accuracy. The specifications contain clauses fixing the maximum impedance on full flow, the limits of accuracy at various rates of flow, and the uniformity of the alloy in the gear train. It also provides for sealing facilities, guarantees as to wear and costs of repair parts; and a special design for the dial cover. The dial cap specified is a hinged glass cover and is designed to avoid the usual trouble of condensation which becomes so dense on the under side of the glass as to prevent reading the dial unless the seal is broken and the dial cap removed. The hinged glass cover opens to an angle of 45°, so that the cover will drop back into position after the moisture has been removed. In order to clean the glass it is not necessary with this arrangement to break the wire seal that passes through the screws. The hinged cover can be sealed in cases where it is considered necessary. This arrangement is intended to be used only with a straight-reading type of register; for it is evident that the hands of a round-reading dial could be so twisted as to be difficult of detection.

Each bidder under the specifications furnishes three sample meters of the smallest size, all of which are tested according to the specifications. One of the samples is submitted to an endurance run and tested for accuracy at fixed intervals of registration. The ⅝ in. size are stopped at 250,000 cu. ft. and the ¾ in. at 375,000 cu. ft. They are then examined and classified as to wear and accuracy. Recently disk meters were purchased under specifications for about half of what was formerly paid for rotary meters.

The policy of the department in installing meters is to place them in basements wherever possible. Whenever the basement is considered unsuitable for such reasons as inaccessibility for reading or repairing, exposure to frost, or a place of business not being adjacent to the building line, the meter is installed in a concrete box beneath the sidewalk or street, preferably the sidewalk. Meters 2 ins. or smaller are installed with brass couplings and sufficient lead pipe to insure flexibility for replacing with another make, they being of different lengths. Meters 3 ins. and larger have flanged connections. In recent years it has been the practice of the department when installing meters 3 ins. or larger to [illegible] about one foot [illegible] the meter and insert between the valve and the meter a 1 in. opening for testing the meter in its service position. [illegible] a large meter that has been in place for a few years, great difficulty is usually experienced in loosening the flanged joint. To break these joints quickly and not disturb the piping, a piece of apparatus has been designed; it consists of a channel beam supported by two posts so that by operating the two draw screws simultaneously a shearing force of 10,000 lbs. can be applied to each joint. From a test, it appears that 120 lbs. per inch of joint surface is amply sufficient. On this basis a force of about 8,000 lbs. per joint will loosen a 6-in. meter, the largest size in general service. In connection with the work of removing and installing large meters, the department has in operation a 1½-ton truck equipped with a chain block and trolley so that one man can handle a meter weighing half a ton. This truck was illustrated and described in ENGINEERING AND CONTRACTING of Feb. 19, 1913. The 4-in. I-beam on which the trolley operates is supported by a framework of 2¼-in. T-bars and extends the entire length and 30 ins. over the rear of the body.

It is the intention of the department to test meters at about certain intervals of registration. These intervals are selected where it is supposed the wear in service will most affect the accuracy. The scheme includes a time limit in which the meter must be tested at least once, even though the registration has not reached the first interval. This time limit is necessary because of the corrosion in meters that have only a small circulation of water. All sizes of meters are tested in the testing department on a Mueller meter testing machine equipped with auxiliary apparatus for testing large meters. It is equipped with a multiple cock so that meters can be tested for accuracy on various rates of flow. In testing on rates of flow greater than 3/10 cu. ft. per minute, a quantity of 10 ft. on the dial is weighed in the tank by the automatic recording scale. The accuracy in per cent when the dial of meter records 10 cu. ft. is then represented by the result of dividing 625 by the number of pounds in the tank. The operator is provided with a diagram, so that, knowing the weight of water in the tank, he can read therefrom the accuracy in per cent. In order to save time when testing on streams 3/10 of a cu. ft. per minute or smaller, only 1 cu. ft. is recorded on the dial. The long machine shown in Fig. 13 is designed to test small meters only, but will test as many as ten at a time. They are arranged in series and held in place by hydraulic pressure in the cylinder at the inlet end of the machine. On this machine a man can easily test meters at the rate of 90 a day, as compared with about 15 a day on the machine formerly used. In testing 3, 4 and 6-in. meters on large rates of flow, the water is measured in a calibrated tank holding 50 cu. ft. It is 30 ins. in diameter, so that each ¼ in. of height represents 1/10 cu. ft. The tank is calibrated only in the vicinity of 10 and 50 cu. ft., and the water level is read by means of glass tubes and scales. In testing by quantities of 100 cu. ft., each ¼-in. calibration represents 1/10 of one per cent.

Before replacing 3, 4 and 6-in. meters they are tested in service by connecting a test meter to the 1-in. valve for that purpose. The operator is provided with a stop watch, so that he can regulate the rates of flow to correspond with the standard shop test as shown in the chart. When such a meter in service shows an error of about 10 per cent or less on the smaller flows, it is regeared to register accurately. When in error more than 10 per cent it is replaced and repaired in the shop by refitting the piston chamber. For repairing and refitting meters a machine shop is now being equipped with such machines as a lathe, pipe machine, drill-press, grinder, shaper, and a drilling and milling machine.

In concluding, it might be mentioned that within the past year and a half the system of operation in the meter department has been changed with the view of handling its work more thoroughly and efficiently. In the system now used, the department replaces defective meters smaller than 3 ins. so as to repair and test them in the shop, after which they are placed in stock ready for service. Also the system of records now in use results in more thorough work as it keeps a check on each man's work. This system is in contrast to the one it has replaced, where meters were repaired in place. The old system proved very unreliable and inefficient. In addition to the present value of the record system, its greatest value is in the future use of the data to determine therefrom the maintenance cost and accuracy in service of the various types and makes of meters. Since this system has been in operation about 80 per cent of the 7,000 meters in service on July, 1911, have been repaired; about 60 tested, and over 14 per cent condemned and replaced with new ones. There are 7,370 meters now in service, which is about 7 per cent of the total number of service connections. The revenue from these metered connections in the past has amounted to about 40 per cent of the total revenue of the water department.

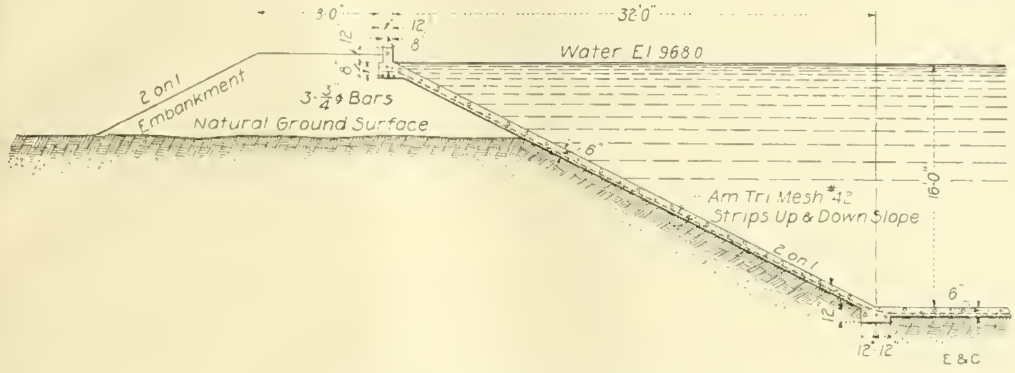


Fig. 1. Cross Section of Reservoir Embankment and Lining for the New Frances-Mary Reservoir at Riverside, Calif.

Some Structural Features of the New Water Supply Works at Riverside, California.

(Staff Article.)

A news article published in Engineering and Contracting of Nov. 12, 1913, outlined briefly the nature and extent of the extensive improvements to the water supply and distribution system of Riverside, Calif., for which contracts were recently awarded. To summarize, the improvements will include 60 miles of water mains, a 3,000,000-gal. distributing reservoir, two booster pumping stations, two high service reservoirs of 500,000 gals. capacity, a 30-in. supply line and the head works. The new supply will be derived from flowing artesian wells about 12 miles from the city. The water will be conducted to an existing reservoir and thence by gravity to the new distributing reservoir. Some of the structural features of the works are here illustrated and described. The article relates primarily to a description of methods specified for laying steel, reinforced concrete and vitrified clay pipe, and for the construction of the three reservoirs and the miscellaneous structures at the head works.

PIPE LINES.

Riveted Steel Pipe.—The pipe will be given a cover of 2½ ft. The top and bottom widths of the trench will be not less than 1½ ft. greater than the nominal diameter of the pipe laid. The pipe will be laid with drive joints, driven not less than 3 ins. or as much farther as may be necessary in order to secure watertight joints. At junctions with existing cross mains and in all other places where it is impracticable to use drive joints, the joints will be made with Dayton couplings or equivalent.

For plugging the dead ends of pipes 8 ins. or less in diameter, painted wooden plugs will be used. These plugs will be of best quality red wood, constructed on a taper, and truly circular in section. The plug will be driven snugly into place and in order to secure it thus lugs will be riveted to the sides of the pipe and a bar, extending over the end of the plug, will be securely bolted to the lugs. Dead ends of pipe larger than 8 ins. in diameter will be plugged by riveting a cast iron flange to the pipe and bolting to it a blind flange, using a ⅛-in. rubber gasket. All plugs or caps will be so installed as to be watertight.

Reinforced Concrete Pipe.—About 17,450 ft. of 30-in. reinforced concrete pipe will be installed. The pipe will be designed to withstand an internal pressure of 20 lbs. per square inch and the external load resulting from a pipe cover of 8 ft. The working stress in the concrete will not exceed 600 lbs. per square inch and that in the steel will not exceed 16,000 lbs. The minimum thickness of 30-in. pipe will be 3½ ins. The circular reinforcement will consist of reinforcing bars rolled spirally or of reinforcing mesh to give the required sectional area of steel. Effective longitudinal bars will also be provided.

The forms for the concrete pipe will consist of cast iron bottom rings and steel plates rolled to a true cylinder. The forms will not be removed until 12 hours after the final set of the cement. As soon as the forms are removed the interior of the pipe will be given a brush coat of neat cement. The pipe will then be covered with burlap and kept wet for at least 16 days. The pipe will not be hauled until it has reached an age of at least 30 days.

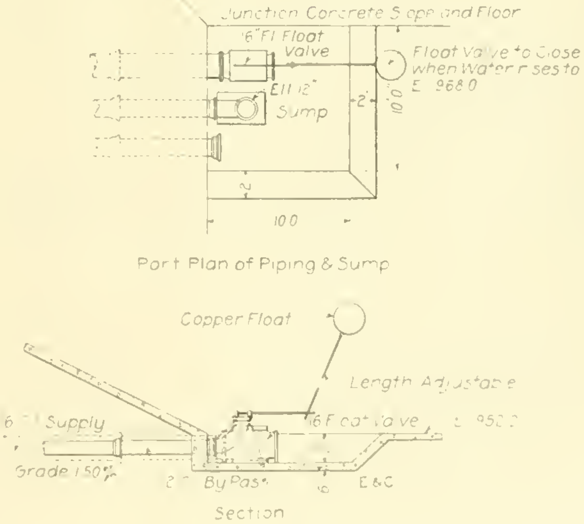


Fig. 2. Plan and Section of Supply Control for the Frances-Mary Reservoir.

Before laying the concrete pipe will be subjected to a hydraulic pressure of 20 lbs. per square inch. One pipe out of each successive set of 100 pipe manufactured will be tested in this manner for 15 minutes. During this test no leakage other than a slight sweating will be allowed. The pipe tested will be selected at random by the engineer.

Vitrified Clay Pipe.—About 6,800 ft. of salt-glazed vitrified clay pipe will be installed for collecting the water delivered by the flowing wells. Since watertightness in the joints of the clay pipe lines is desirable the method specified for ensuring it is of interest. As a cementing medium for the joints "G-K" compound, or equivalent, will be used. A gasket of clear oakum or jute will be calked tightly into place to a depth just sufficient to leave 1 in. for the jointing compound. An asbestos runner will be clamped around the pipe at the bell, leaving a triangular gate at the top through which to pour the compound.

The jointing compound will be heated in an iron pot of sufficient size to hold material for jointing 10 or more joints. It will be continually stirred while heating and, when in a state as liquid as water, will be poured into the joint through the open space between the runner and the bell. A pouring vessel large enough to completely make one joint at a single pouring will be used and the runners will not be removed until the compound has thoroughly set. Jointing of pipes on the bank of the trench will not be allowed unless special permission to do so is granted by the engineer.

HEAD WORKS.

Well Houses.—Well houses will be constructed over some of the wells. These houses are 6 ft. 4 ins. by 5 ft. in plan. The floor and walls are of plain concrete 6 ins. thick. The walls are 4 ft. 6 ins. high. The roof is made of 1-in. tongue and groove lumber on 2x4-in. framing. The ridge of the roof is set 8 ins. off center and the short side of the roof is hinged for a door which extends the full width of the house. The well casing extends 18 ins. above the floor of the house. After emerging from the well the water passes over a rectangular weir 18 ins. long and 8 ins. deep, thence leaving the house through a vitrified clay collecting pipe. The portion of the house surrounding the well casing—that is, up-stream from the weir, is to be flushed out at intervals through an 8-in. sluice gate the bottom of which is at the floor level.

Sand Box.—A sand box is situated at the down-stream end of the collecting system. It is 4x6 ft. in plan and 6 ft. deep. It is constructed of plain concrete with walls 8 ins. thick and the floor 6 ins. thick, and is covered with a reinforced concrete slab 6 ins. thick. The vitrified clay collecting pipe is 24 ins. in diameter at this point and the invert of the pipe is 2 ft. 6 ins. above the floor of the box. The water is made to pass under a concrete baffle wall the lowest part of which is 2 ft. above the floor. The sand deposited in the box is flushed out when necessary through a 10-in. sluice gate at the downstream end of the box.

Equalizing Chamber and Register House.—At the down-stream end of the main collector an equalizing chamber and register house will be constructed. This structure will be 10 ft.

[illegible]

[illegible]

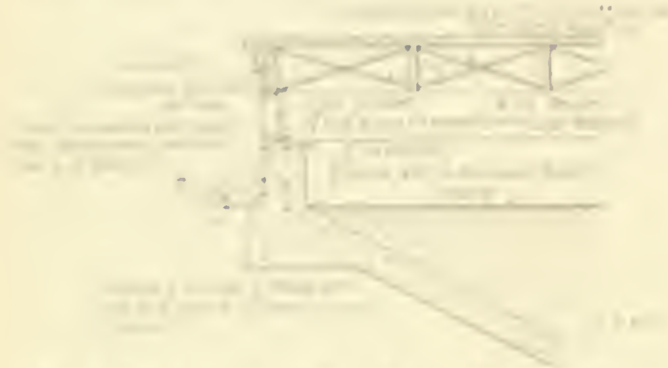


Fig. [illegible] Part Section of Roof and Walls of Frances-Mary Reservoir

[illegible]

[illegible]

[illegible] with a [illegible] along the [illegible] operations [illegible] embankment is [illegible]

[illegible]

It is intended [illegible] supply line to the Mary-Frances Reservoir will be one of the first mains to be installed and as soon as this supply line is completed and ready for operation [illegible] will be furnished free by the city for use in wetting down the reservoir embankment. In case such supply is not available the city will [illegible] convey water to the site for the above use and the contractor will provide the necessary storage [illegible] supply [illegible] the rate [illegible] insufficient or [illegible]. The contractor will [illegible] the [illegible]

Waste Material.—[illegible]

[illegible]

Before beginning the excavation for any reservoir all vegetation and vegetable matter will be removed from the area of the work.

The use of triangular mesh reinforcement in the reservoir lining is illustrated in Fig. 4.

Before attempting to set a strip of reinforcing mesh the same will be hammered flat and straightened out in order that it may be set as nearly as possible in the required position. The strips of mesh will be lapped 6 ins. longitudinally and ½ in. transversely, and will be wired together at 6 in. intervals.

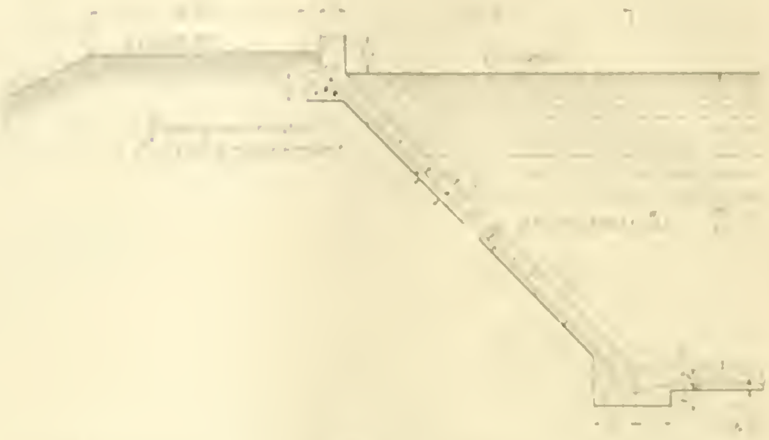


Fig. 4. Typical Section of Reservoir Wall and Lining of New Van Buren Reservoir at Riverside, Calif.

Concrete Work.—In advance of placing any concrete in floors or of lining the surface of any excavation, the same, if in earth, will be accurately graded, moistened and thoroughly tamped. Such concrete work will be carried on in strips parallel to the direction in which the reinforcing mesh is laid. The concrete will be compacted with shovels and the surface will be given a smooth troweled finish. Just before stopping work at any time, the edge of the concrete will be sloped off on approximately a "two to one" slope and the edge of the reinforcing mesh will be allowed to project to form a bond into the new concrete

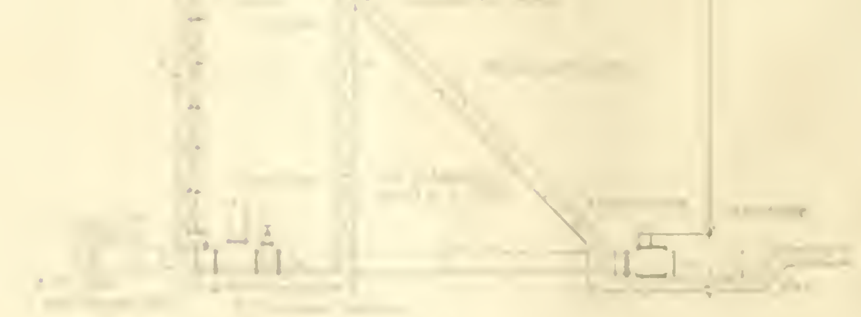


Fig. [illegible] Section Showing Control Works for Van Buren Reservoir

The contractor will install and adjust for proper operation the float valve, shown in Fig. 5, so as to close the supply main when the water in the reservoir rises to a depth of 15 ft. above the floor.

The roof construction of this reservoir is shown in Fig. 3. The beams shown rest upon the tops of reinforced concrete columns which are spaced, approximately, at the corners of 15 ft. [illegible] squares. Each column is 10 ins.

quare and is reinforced with four ¼-in. ound bars. These vertical rods are held in ace by No. 8 wire ties spaced 12 ins. on centers.

HIGH SERVICE RESERVOIRS

The two high service reservoirs, known as e Van Buren and Elsinore reservoirs, are ach of 500,000 gals. capacity. The bottom of e former is 91x25 ft. in plan and that of the tter is 18 ft. x 98 ft. The depth of water the Van Buren is to be 15 ft. and in the lsinore 18 ft. The side slope of the lining the former is to be 1 on 1 and in the latter on 1.

The contour plats for these reservoirs indicate that they will be located almost entirely in excavation. However, at some of the corners it will be necessary to compact a small amount of embankment against which the reservoir lining will be constructed. The requisite tamping will be performed by hand with suitable tools as directed by the engineer and will be paid for at the rate stipulated in the proposal. A cross section of the Van Buren reservoir lining is shown in Fig. 4, and the controlling apparatus is shown in Fig. 5 of the high service reservoir.

The exterior exposure of the roofs will be the same as described for the Mary Frances Reservoir, and all exposed lumber inside and outside will be surfaced and given two coats of the best lead and linseed oil paint.

Class "B" concrete will be used throughout the construction of these reservoirs. This concrete will consist of 1 part cement and 6 parts of aggregates, and will contain a waterproofing compound. Batch or hand mixing is specified. The waterproofing will be used in an amount sufficient to satisfy the manufacturer's specifications.

After the reservoirs have stood approximately full of water for three days, tests of the same will be applied. They will be filled with water to their high water marks, all outlet valves will be closed and they must show not more than 1 in. each in depth of leakage in 24 hours. In case of failure of any reservoir to pass the above test, the water will be withdrawn from the same and such further precautions will be taken to make it watertight as will accomplish the desired result.

The engineering work for the extension of the water works at Riverside was done by Burns & McDonnell, Consulting Engineers, of Kansas City, Mo. The information presented in this article is taken from the engineers' plans and specifications.

Bridges

esign and Construction of the St. Croix River Bridge at Hudson, Wis.

(Staff Article.)

A somewhat novel design was adopted for e piers of the highway bridge over the St. roix River at Hudson, Wis. On account f the required clearance of 53 ft. above igh water it was necessary to choose an onomical type of pier construction to keep e cost within reasonable limits. The type ecided upon consists of two slender reinrced concrete cylinders cross-braced at two oints. The total length of the main structure is 986 ft. It consists of seven light 90-ft. riveted deck spans and one light 136-ft. veted through span, with an 18-ft. clear oadway (see Fig. 1). To the east of the ans shown in Fig. 1 there is a trestle approach 220 ft. long. The type of construction dopted for the first 180 ft. of this approach onsists of a roadway supported on steel ents, spaced 20 ft. apart. The remaining 40 ft. consists of braced reinforced concrete retaining walls and an earth roadway. The idge has a latticed railing throughout its ntire length. The portion of the bridge west f the through span is on a 7 per cent grade, e through truss is level, and the east portion is on a 1.5 per cent grade. The structure designed as a "Class B Bridge," Cooper's pecifications, except that provision was made r a future concrete floor.

TRUSSES

Deck Spans.—The 90-ft. riveted deck ans are of the subdivided Warren type, ith nine 10-ft. panels and a depth of 10 ft. he clear roadway is 18 ft., and the trusses re spaced 13 ft. 6 ins. apart. The center ection of the upper chord consists of two 8x8x⅞-in. angles and the end sections of wo 8x8x⅝-in. angles. The center section of the lower chord consists of two 8x8x⅝-in. angles and the end sections of two 8x8x½-in. angles. The web members are composed of two angles, the connections being made with ⅜-in. gusset plates. The floor construction is as follows: 12-in. 31½-lb. I-beam floorbeams; nine lines of 3x10-in. wooden joists, and a 3-in. plank floor. A light system of sway and portal bracing is used.

Through Span.—The 136-ft. through truss span consists of eight 17-ft. panels. The trusses, which are spaced 20 ft. apart, have curved upper chords and a maximum height of 27 ft. at the center. The upper chord section consists of two 9-in. 13¼-lb. channels and a 15x¼-in. cover plate. The center section of the lower chord is composed of four 4x4x⅜-in. angles and the end sections of four 4x4x5/16-in. angles. The posts consist of four angles and the diagonal tension members are made either of two angles or two channels. A light system of sway and portal bracing is used. The upper and lower laterals consist of square rods. The floor construction is as follows: 20-in. [illegible] I-beam floorbeams; two outside lines of 8-in. 11¼-lb. channels and [illegible] lines of [illegible] I-beam joists, and a 3-in. plank floor.

APPROACH SPANS

Steel Trestle.—The 180 ft. of trestle approach nearest the truss spans consists of nine steel bents spaced 20 ft. apart, supporting an 18-ft. clear roadway. The first and fifth panels only have longitudinal bracing.

The posts of the trestle bents have a batter of 1 in. in 12 ins. and have channel angle bracing. These posts rest on concrete pedestals, which are supported on a reinforced concrete strut extending between the bases of the posts. This strut, which also acts as the footing, has a cross section of 1½x1½ ft. and a length which varies with the height of the bent.

A section through the roadway showing a trestle bent and the floor construction is given in Fig. 2. This drawing shows the type of construction, the sections and the general dimensions.

Retaining Wall Construction.—The east 40 ft. of the approach consists of an earth roadway held in place by means of reinforced concrete retaining walls. These walls are connected at intervals of 10 ft. with reinforced concrete ties. The type of construction adopted for this part of the approach is shown in Fig. 3. This drawing also shows the manner in which the thrust against the retaining walls is resisted.

REINFORCED CONCRETE PIERS

The type of construction adopted for the piers consists of two reinforced concrete columns, with vertical and spiral reinforcement, battered together near the base and near the top, and joined at these points by reinforced concrete diaphragms. The posts are enlarged near the water line, and are supported on pile foundations.

Piers for Through Truss.—Piers Nos. 6 and 7, which support the through truss (see Fig. 1), have [illegible] diameter cylinders on [illegible] diameter footings. The height of these piers above the footings is [illegible], and the height of the footing is 10 ft., making a total height of [illegible]. The cylinders are spaced 21 ft. on centers. Figure 4 shows an elevation, a pile plan and sections of these piers. These drawings clearly illustrate the type of construction and the position and size of the reinforcement. The pedestals of the through truss span rest on the tops of the pier shafts, while those of the adjacent deck spans are supported on the cross girder, as shown in Fig. 1. The heights of these piers were increased 5 ft. over that originally planned, after construction had started, due to a change in the required clearance.

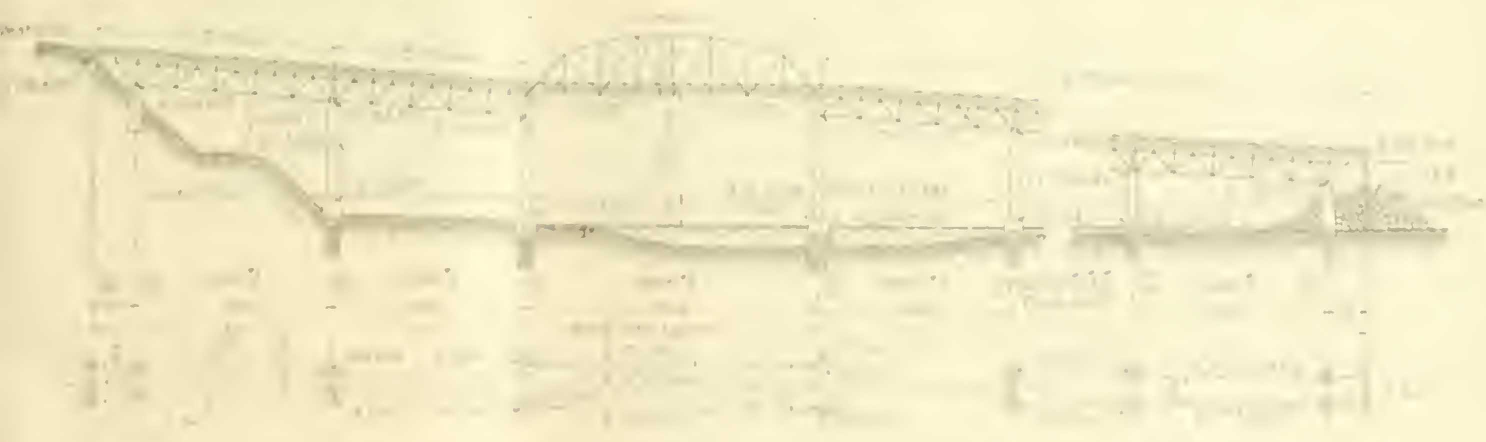


Fig. 1. Pier Plan and Elevation of Main Spans of St. Croix River Bridge at Hudson, Wis.

Piers for Deck Spans.—Piers Nos. 2, 3, 4, 5 and 8 (see Fig. 1) for the deck spans are of similar construction to those shown in Fig. 4, but have different heights and sections. The pier shafts for these piers are spaced 13 ft. 6 ins. on centers. Piers Nos. 2, 3, 4 and 5 have 4-ft. diameter pier shafts, while pier No. 8 has a 4-ft. 6-in. diameter shaft. Each of these piers have 8-ft. diameter footings supported on ten piles, the height of these footings being 12 ft. The elevation of the tops of the footings for piers Nos. 2, 3, 4 and 5 is 104.0 ft., and that of pier No. 8 is 111.0 ft. As the bridge is on a grade the elevations of the tops of the piers vary, these elevations being 131.029 ft. for pier No. 2, 137.329 ft. for pier No. 3, 143.629 ft. for pier No. 4, 149.929 ft. for pier No. 5; and 162.529 ft. for pier No. 8. Owing to a change in the required clearance over the main channel the heights of these piers was increased 5 ft. after construction had started. In providing for this additional height the rods for the added section of the piers were lapped 3 ft. over the old rods. The lengths of these pier shafts above the footings therefore are 27 ft. [illegible] for pier No. 2; 32 ft. 1 in. for pier No. 3; [illegible] ft. 7½ ins. for pier No. 4; [illegible] for pier No. 5, and 51 ft. [illegible]

The upper [illegible] of the pier shafts [illegible] and [illegible] lower [illegible] 1:2:4 [illegible] the [illegible] a [illegible] in Fig. 4.

[illegible]

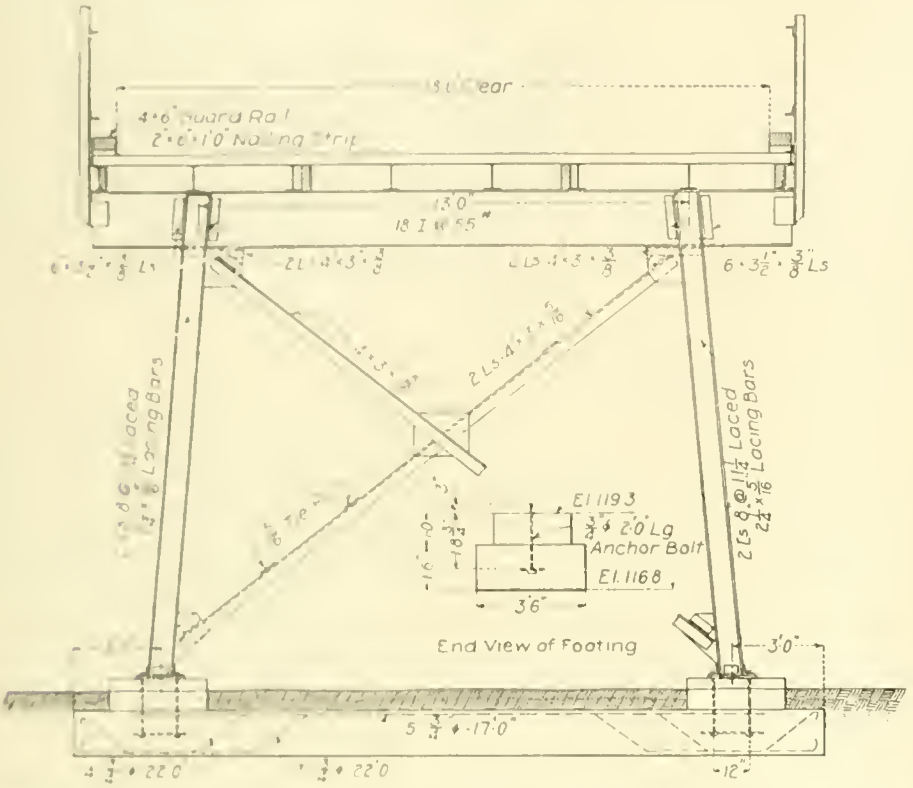


Fig. 2. Elevation of Steel Trestle Bent Showing Type of Construction for Bent and Footing.

CONSTRUCTION FEATURES.

Figure 5 shows part of the construction equipment for the piers. The hoisting engines and elevating tower for spouting the concrete were placed on scows, the tower being anchored to the adjacent piers. The view shows some of the pier reinforcement and forms in place.

Figure 6 shows the equipment used for erecting the through truss span. It will be noted that two additional struts are provided to give more rigidity to the stiff-leg derrick. The trusses were partially assembled at the site, and were hoisted in place by the derrick, being held in position by guide lines. The [illegible] the portion which was assembled on the ground, and also that erected on falsework.

Figure 7 shows a picture of the completed structure. It will be noted that a pile and timber protection has been constructed for the piers supporting the through truss span. It is evident that a collision of a boat with these [illegible] piers might result disastrously.

The bridge was designed and the detail plans [illegible] by C. A. P. Turner, Minneapolis, Minn. The Central States Bridge Co., Indianapolis, Ind., were the contractors for all the substructure and superstructure, and made the detail plans subject to the approval of the designer. The latter also fabricated the steelwork, with the exception of the steel approach, which was fabricated and erected by the Minneapolis Steel & Machinery Co., Minneapolis, Minn. We are indebted to Mr. Turner for the principal data contained in this article.

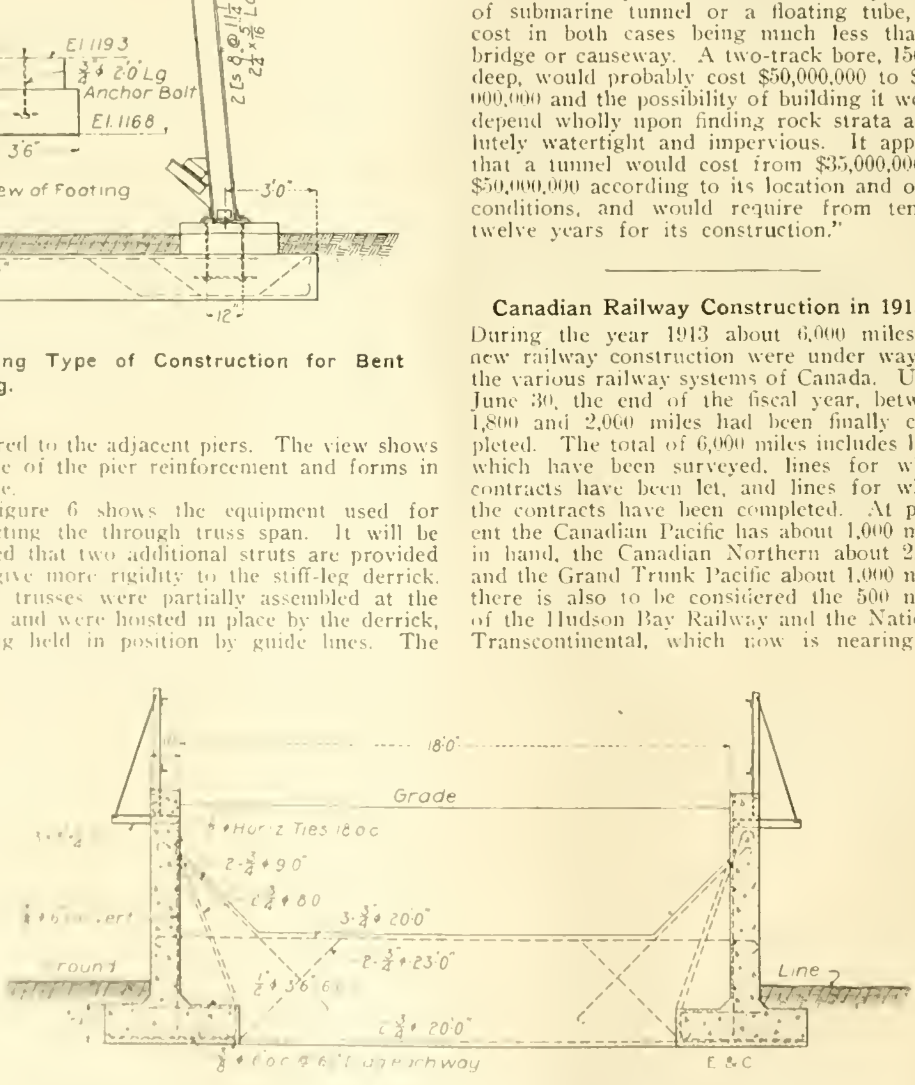


Fig. 3. Section of Reinforced Concrete Retaining Wall Approach, Showing Type of Construction and Reinforcement.

Tunnel to Connect England and Ireland.

—An American engineer, Henry Grattan Tyrrell of Chicago, has proposed a practical plan for connecting England and Ireland by a channel railway, and the feasibility and value of such an enterprise is being widely discussed. The location indicated by Mr. Tyrrell is that from Whitehead on the Irish coast, northeast of Belfast, to Port Patrick, a distance of about 23 miles. Mr. Tyrrell is quoted as saying:

"There are at least four possible methods of establishing a railway across the channel: (1) a continuous embankment or causeway; (2) a submarine tunnel; (3) a submerged floating tube or viaduct and (4) a bridge. Investigations show that the best way of establishing a channel railway to Ireland is either by means of submarine tunnel or a floating tube, the cost in both cases being much less than a bridge or causeway. A two-track bore, 150 ft. deep, would probably cost $50,000,000 to $60,000,000 and the possibility of building it would depend wholly upon finding rock strata absolutely watertight and impervious. It appears that a tunnel would cost from $35,000,000 to $50,000,000 according to its location and other conditions, and would require from ten to twelve years for its construction."

Canadian Railway Construction in 1913.

—During the year 1913 about 6,000 miles of new railway construction were under way on the various railway systems of Canada. Up to June 30, the end of the fiscal year, between 1,800 and 2,000 miles had been finally completed. The total of 6,000 miles includes lines which have been surveyed, lines for which contracts have been let, and lines for which the contracts have been completed. At present the Canadian Pacific has about 1,000 miles in hand, the Canadian Northern about 2,000, and the Grand Trunk Pacific about 1,000 miles there is also to be considered the 500 miles of the Hudson Bay Railway and the National Transcontinental, which now is nearing its completion. The present activity in construction work constitutes a record.

U. S. Bureau of Labor Safety.—The committee on labor of the U. S. House of Representatives has made a favorable report on the creation of a Bureau of Labor Safety and a bill for this purpose will be introduced in Congress. The new bureau will be a part of the Department of Labor. Under the provisions of the bill the Secretary of Labor will be empowered to engage engineers and architects, with a view to securing expert advice on industrial matters.

Some Erection Features of the New Quebec Bridge.

The designers of the new Quebec Bridge have had constantly in mind a structure which easily could be erected. The construction features have been carefully worked out, and no detail of design has been adopted which would involve large unknown stresses during erection. Secondary stresses have been largely avoided by using pin connections where such stresses would ordinarily exist. An elaborate wooden model of the anchor and cantilever arm and of the erection traveler has been constructed, using a scale of ¼-in. = 1 ft. It is possible by means of this model to anticipate many of the erection difficulties and to make provision for them. As we have recently visited the site of the bridge we shall give some general data relative to the location and to the erection progress. Most of the data relative to erection equipment have been taken from an article by H. F. Stratton in the December, 1913, issue of the Engineering Magazine. We have described some of the main features of design of the new Quebec Bridge in ENGINEERING AND CONTRACTING of Oct. 1, 1913.

GENERAL DESCRIPTION.

The new Quebec Bridge is located at Neilsonville, at the site of the old structure, which is about eight miles from Quebec. The bridge has a center span of 1,800 ft., consisting of a suspended span of 640 ft. and two cantilever arms of 580-ft. span each; two anchor arms of 515-ft. span each; and approach spans, the total length of the structure being 3,240 ft. The new bridge is much heavier than the old structure, which made it impossible to use the old piers. The new piers for the main span are each 67 ft. south of the location of the old piers. The bridge has a clearance of 150 ft. above high water.

The new bridge was designed and is being constructed by the St. Lawrence Bridge Co., which was organized to build this structure. The designing office is in Montreal, and the bridge shop is about five miles from Montreal, being about one mile from the Dominion Bridge Co.'s plant at Lachine Locks. The capital stock of the St. Lawrence Bridge Co. is held equally by those interested in the Dominion Bridge Co. and in the Canadian Bridge Co., and many of the officers of the new company are former employees of the latter companies. The bridge shop of the St. Lawrence Bridge Co. is a new structure, and was designed expressly to build the new bridge. It contains considerable special machinery designed to fabricate the large sections of the new structure. The plant was built at a cost of about $1,000,000, and must be paid for out of the profits of the new Quebec Bridge.

The St. Lawrence Bridge Co. has an elaborate equipment at the bridge site. The various bridge sections are unloaded from the cars by means of a large traveling crane, and are transported to the bridge site over the company's system of tracks. There is now stored at the site a considerable amount of fabricated material, principally for the floor system. The piers and abutments are now completed and the approach spans are in place. For constructing the piers at the north end of the bridge an inclined railway was built on each side of the north abutment. The material for the piers was transported to the site, where it was transferred to the inclined railway, which in turn carried it to the location of the pier. The material was then picked up by derricks and set in place.

ERECTION TRAVELERS.

Each of the cantilever arms is to be erected as a separate structure, with its individual erection equipment. The center span, which will weigh about 6,000 tons, will be assembled on barges near the bank of the river. It then will be floated into position, and by hydraulic power will be raised to its final position in the structure.

Each erection traveler for the cantilever arms is constructed of steel and is about 200 ft. in height. The traveler is carried by four trucks having six wheels each. At its base it spans two railroad tracks, on which are transported the cars containing the material for the bridge. The total weight of each of these travelers is about 840 tons, although an attempt was made to make this weight a minimum by constructing the crane girders which project over the traveler of nickel steel.

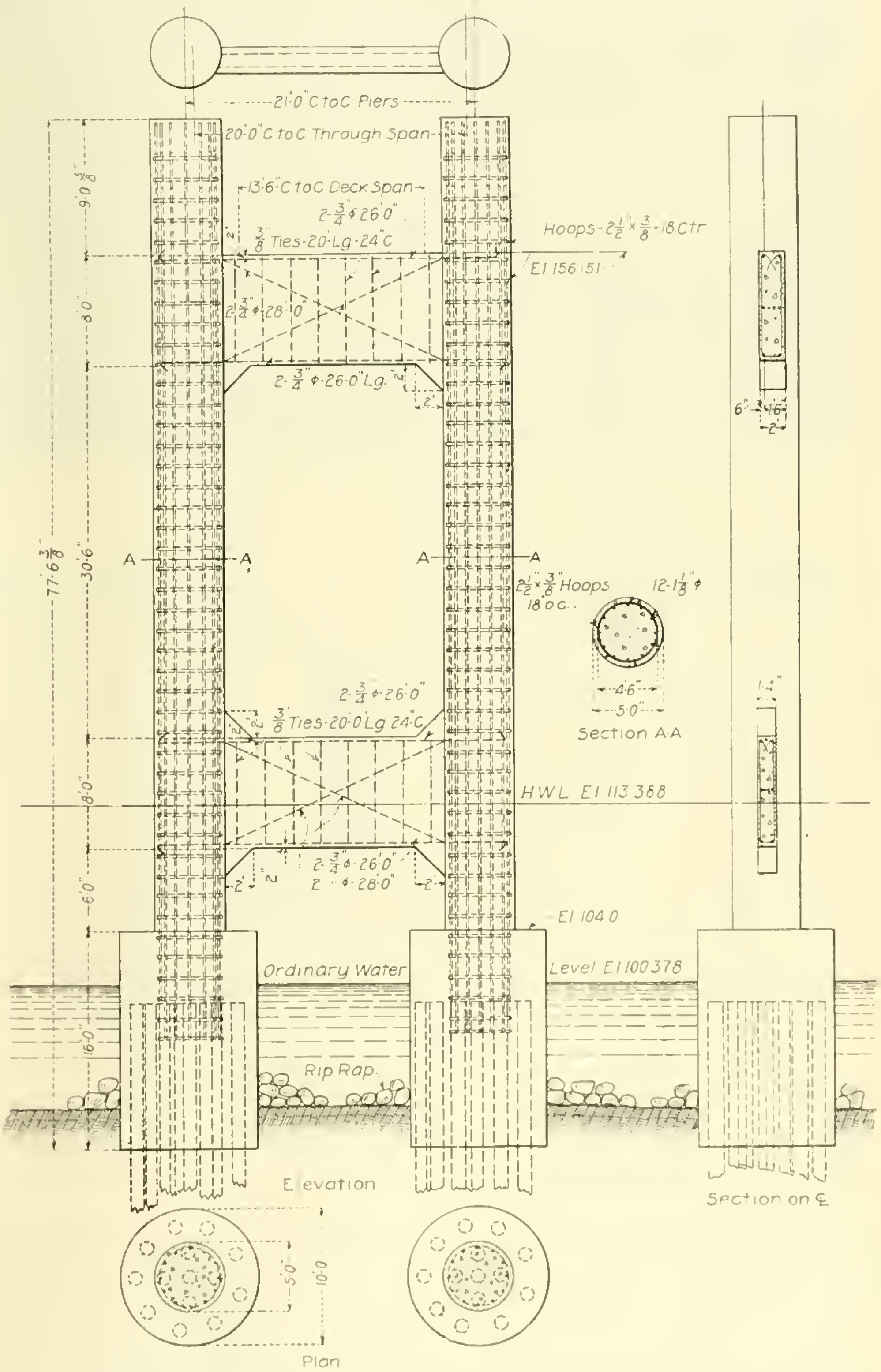


Fig. 4. Elevation, Plan and Sections of Reinforced Concrete Piers, Nos. 6 and 7, Showing Type of Construction and Reinforcement.

The hoisting equipment of each tower is both elaborate and diversified. On each of the four corners there is a 90-ft. 15-ton derrick, and on top of the traveler there are two traveling cranes, each crane carrying two 55-ton main hoists and two auxiliary gantry cranes (see Fig. 1). On each auxiliary gantry crane there are two 5-ton hoists. All motions are motor-driven, except the bridge travel of the auxiliary gantries. The main traveler will be moved along the cantilever arms by using, temporarily, two of the 50-HP. motors of the 15-ton derricks. Table I shows the distribution and capacity of the motors for one traveler. It will be seen that 26 motors, with a total horsepower of 752, are used for hoisting operations.

On each side of the river, high tension hydroelectric power is stepped down by transformers, and is converted into 250-volt, direct current at two sub-stations, one on each side of the river. These sub-stations are supplied by separate systems, and are connected by submarine cables. Each sub-station is of sufficient capacity to operate all the motors on both sides of the river, including those required for the air compressors, which requires about 1,000 HP.

Direct current is used exclusively on the erection towers, as it affords better control than alternating current, and permits the ready employment of dynamic braking for the lowering machinery. The lowering of some of the heavy members used in the construction of the cantilevers would generate an excessive amount of heat in any type of friction or mechanical brake. By dynamic braking is meant that the motors are temporarily converted into generators during the lowering operations, the different speeds being obtained by inserting or removing a series of resistance. This enables most of the heat to be dissipated in a cast grid resistance, which is located in the base of the main traveler.

TABLE I—MOTOR EQUIPMENT FOR ONE MAIN TRAVELER.

Type	Service of motor	No. of motors.	Horse-power each motor.
15-ton derrick	Hoist	1	50
	Swing	1	[illegible]
50-ton main traveler	Hoist	1	80
	Bridge	[illegible]	16
	Trolley	4	5
[illegible]-ton auxiliary electric	Hoist	8	2[illegible]
Total		26	75[illegible]

The retarding effect of the dynamic braking action, as applied to the lowering of a load, exists, practically, only while the motor armature is rotated. It therefore requires a separate means to hold the load stationary. For this purpose a magnetic brake is placed on the armature shaft to lock the motor when the controller is brought to the "off" position. On some of the hoist motors safety devices are provided to stop the motor automatically if the operator should allow the load to overtravel. A large switchboard is installed in each main traveler and numerous push button stations at points easily accessible to the operators, by which they can open or close the motor circuits.

METHOD OF ERECTION

The two main trusses are to be erected symmetrically on each side of the main traveler. It is planned first to erect the north anchor arm over falsework, using one of the travelers. After this arm has been erected the south anchor span will be erected by using the second traveler. At the same time that the south anchor arms is being erected the first traveler will be at work on the north cantilever arm, and this will be finished at about the same time that the south anchor arm is completed. The first traveler will then be taken down and reassembled at a point about two miles below the bridge site, where it will be used for erecting the suspended span on barges. In the meantime the second traveler will be at work erecting the south cantilever arm, and it is planned to complete the erection of this arm and the suspended span (on barges) at the same time. The suspended span then will be transported to the bridge site and raised into place.

An operating platform will be built on each traveler at a height sufficient that a clear view can be secured of the material as it comes from the flat cars and is elevated into its final position. A house will be built in the center of this platform to protect the control apparatus, except that portion which is directly operated by the workmen—this will not be enclosed. Six men will be stationed on the operating platform and two men on each of the main cranes at the top of the traveler. To secure the proper co-operation between the widely separated workmen, a telephone system will be installed with a central switchboard and permanent connections with the shore. An electrically operated passenger elevator will serve the different levels of the traveler.

The following is a brief description of the method of erecting any typical part of the structure: The member first is brought beneath the traveler on flat cars, and is elevated and placed by two, or possibly four, of the

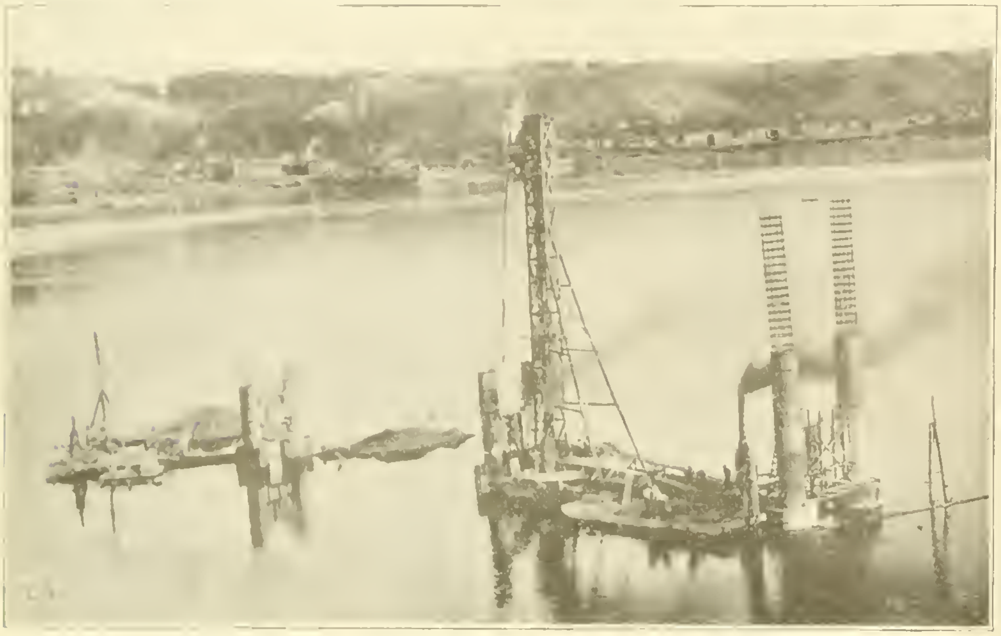


Fig. 5. View Showing Construction Equipment for Reinforced Concrete Piers.

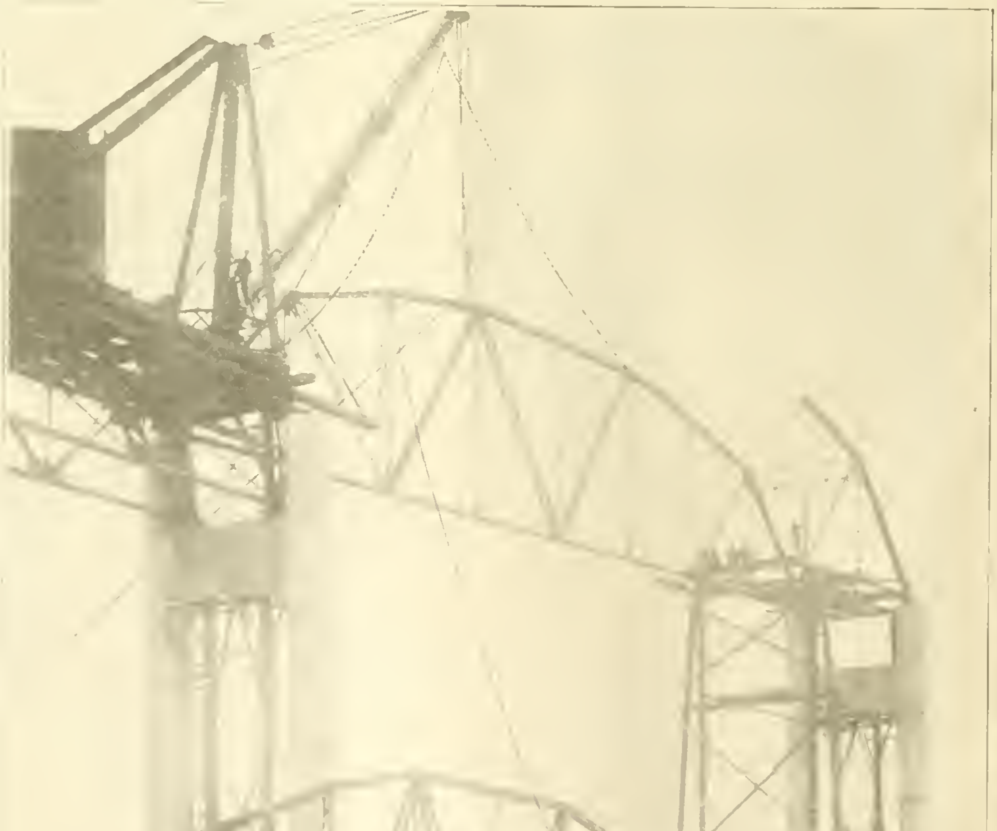


Fig. 6. View Showing Construction Equipment and Method of Erecting Through Truss Span.

55-ton main hoists. The pin is then brought into position by one of the five-ton auxiliary hoists. A large ram for driving the pin is then brought into position by another five-ton auxiliary hoist, and the pin is driven into position.

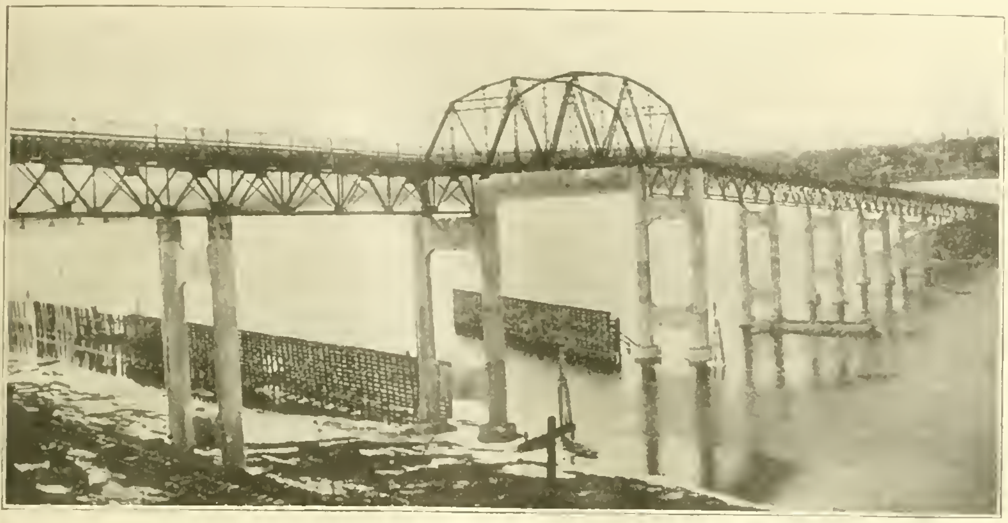


Fig. 7. View Showing East Portion of Completed Bridge—View Also Shows Pier Protection.

It is expected that work can be prosecuted only for about eight months of the year. With this condition in mind it is expected that the bridge will be completed in 1918.

Highway Engineering Course at Columbia University.—Six illustrated lectures on highway engineering are announced by Columbia University for the month of January, as follows:

Jan. 9—"Commercial Mining, Transportation and Storage of Asphaltic Oils." William H. Kershaw, Chief Engineer, Paving and Roads Division, The Texas Company, New York City. Jan. 15—"Organization and Methods of Street Cleaning Departments." William H. Connell, Chief, Bureau of Highways and Street Cleaning, Philadelphia, Pa. Jan. 20—"Gravel and Its Use in Highway Construction." Paul D. Sargent, Chief Engineer, State Highway Commission, Augusta, Me. Jan. 23—"Details of Construction of Car Tracks in City Pavements." George W. Tillson, Consulting Engineer to the President of the Borough of Brooklyn, New York City. Jan. 24—"European Rock Asphalts: Obtaining, Preparation and Uses." J. W. Howard, Consulting Engineer, New York City. Jan. 29—"Plant, Highway and Laboratory Inspection of Bituminous Materials." Francis P. Smith, Chemical and Consulting Paving Engineer, New York City.

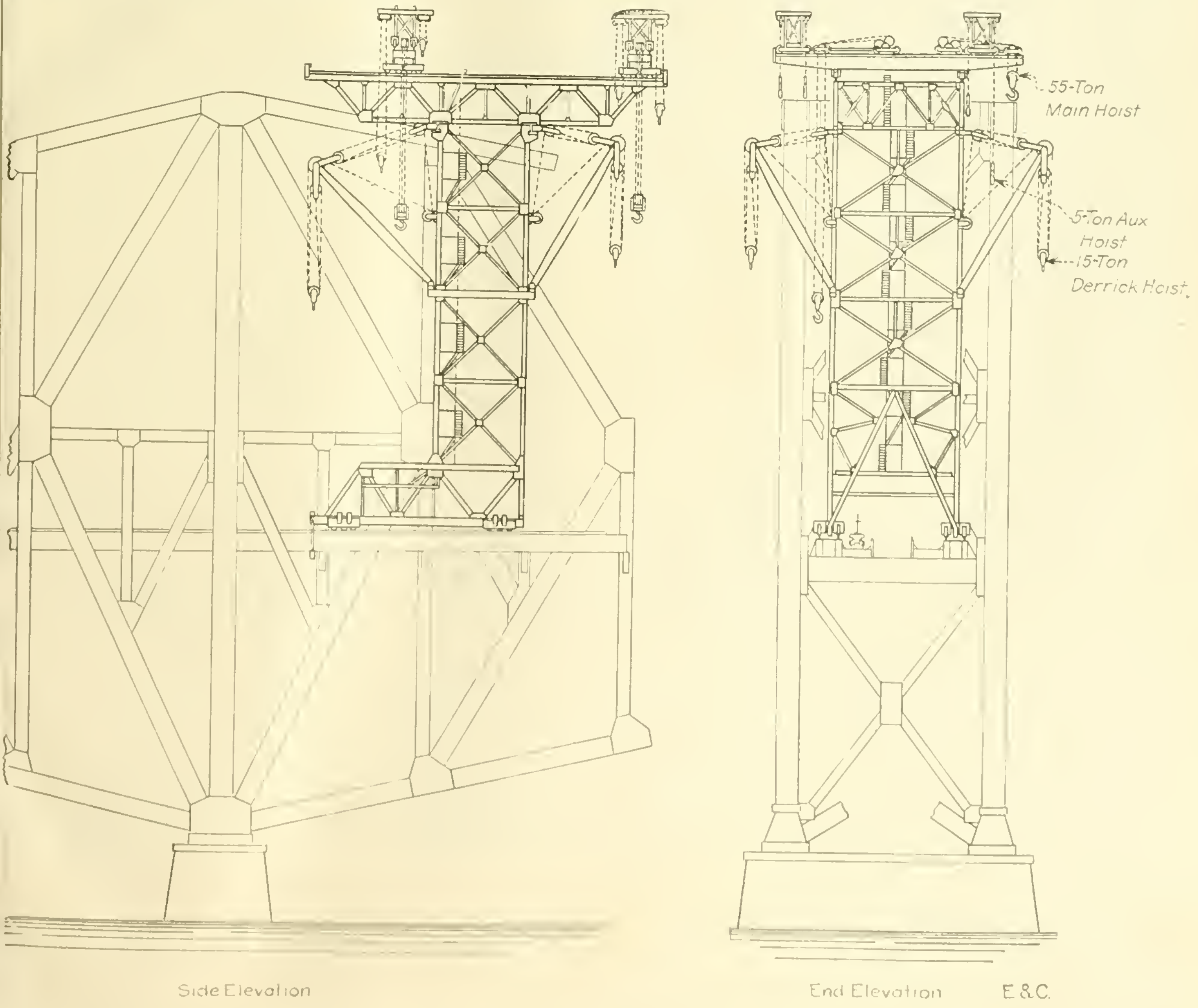


Fig. 1. Side Elevation and End Elevation of Erection Traveler for New Quebec Bridge, Showing Type of Traveler and Location of Motor Hoists.

Sewerage

Results of a Study Made to Determine the Most Economical Motive Power for Pumping Sewage at Lynn, Mass.

Contributed by Frank H. Carter, Assoc. Mem. Am. Soc. C. E., Designing Engineer, Cambridge, Mass.

In making a study for the disposal of sewage for the city of Lynn, Mass., under the direction of Chas. W. Gay, Consulting Engineer, one of the problems with which the city had to contend was the selection of the proper motive power; i. e., oil, steam or electric power, particularly with respect to economy.

So little information is available concerning maintenance charges on oil engines that the attention of the writer was first directed to this feature. The low fuel cost of crude oil engines has long been a matter of attraction to engineers engaged in studying similar problems, but the rumors of short-lived engines and high maintenance charges have discouraged some of the investigators in the past.

That these rumors have been founded in many cases on some basis of fact, there can be little dispute. Past American practice in oil engine construction has not been altogether to the credit of the American manufacturers. There has been a marked improvement of late, however, in the construction of several of the leading makes and this is one of the encouraging facts of oil engine practice.

To substantiate or to disprove the rumored high maintenance costs and the short lives of the engines, the author of this article was commissioned to visit several of the larger installations of crude oil engines in Massachusetts and in Pennsylvania. Additional data on the cost of maintenance and depreciation of unquestioned reliability and accuracy were furnished by the engineers of one of the bonding companies operating a large electric light and power company at a point some distance from Boston.

Briefly, the facts gleaned during the tour of inspection concerning the operation maintenance—costs and depreciation of these various machines, without going too much into detail, are as follows:

Plant No. 1.—Two engines of the so-called low pressure type were in operation at the time of the visit. These had been through a fire which had burned down the plant over them one fall, and the plant had been allowed to remain without attention outdoors in the ice and storms of a winter, the engines also plainly showed the warping due to the fire. The engines had been running some years previous to the fire and had been in daily operation since rebuilding. The operator stated that the cost of repairs was very small. A newer type of this same make, however, was apart for repairs. With this newer "high pressure" type there had been some trouble. The bearings had been "heating," and the operator and superintendent both agreed that the machine was not lined up properly, even at the start when first erected.

TABLE I.—ESTIMATED FUEL CONSUMPTION IN OIL ENGINES.

	High pressure.	Low pressure.
Full load	0.48 lb. HP.-hr.	0.80 lb. HP.-hr
¾ to ½ load	0.52 lb. HP.-hr.	0.90 lb. HP.-hr
½ to ¼ load	0.60 lb. HP.-hr.	1.08 lbs. HP.-hr

It appeared to the writer that this machine was not heavy enough to stand the work. High pressure results in great saving of fuel, but with this a consequent increase in maintenance and lubricating charges.

Plant No. 2.—The engine was a comparatively large machine of one of the later types, and, though it had been in operation but a short time, seemed to be giving good satisfaction and was said to be costing very little for repairs. The operator spent only a part of his time in the room, being required to attend to other duties in the factory some distance off.

Plant No. 3.—The fact that the operator could sleep soundly during the running of the engine apparently was one of the assets of this machine, according to the superintendent of the plant. The engine was one of the old low pressure types and had cost little or nothing for repairs. A common laborer of perhaps rather more than the average degree of intelligence was the operator.

Plant No. 4.—This engine, which was of what might be called the medium pressure type, was used to pump water and was operated by an engineer who had previously had charge of the steam plant, had cost very little for repairs, but had been installed only for a short time. The one complaint of the operator here was that the engine, pump and force main ought to have been larger so that his hours of service each day might have been shorter.

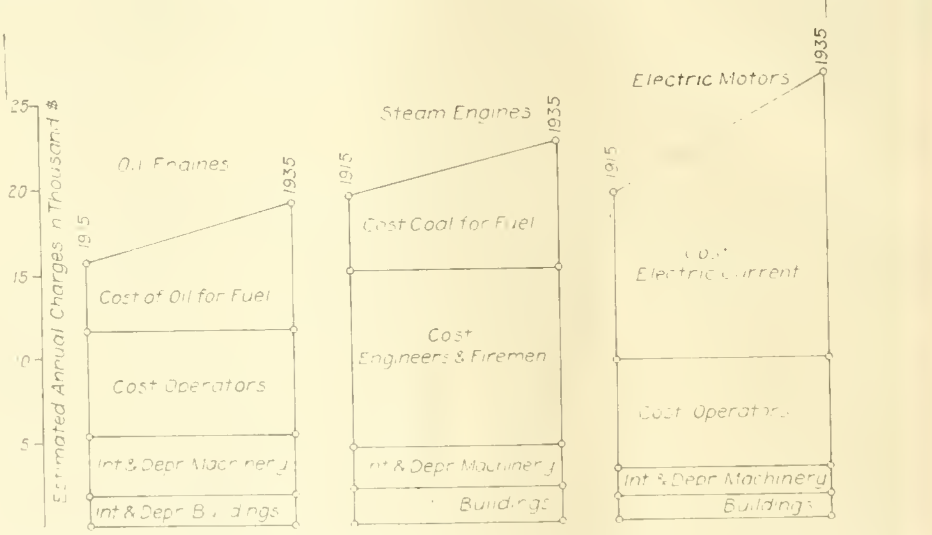


Fig. 1. Comparative Estimated Annual Fuel, Labor, Interest and Depreciation Charges for Oil, Steam and Electric Motive Power for Pumping Sewage at Lynn, Mass.

Plant No. 5.—This plant was a very large installation of oil engines where steam was used elsewhere as a motive power. Two types were in evidence and there were several machines of each, some of the very high pressure and others of the low pressure of another make. The best of machine shops was an integral part of the factory, consequently repairs were at a minimum. The guide who piloted us about the work was one of the superintendents in charge of motive power and was apparently intelligent, unprejudiced and fairminded. On pressing him as to the reason for the purchase of so many oil engines in the midst of so much more first-class steam power, he said that there was no very good reason for the purchase in this particular case and if the power were to be sought now, additions would undoubtedly be made to their steam plant.

TABLE II.—COST TO RAISE WATER 1 FT. HIGH, METROPOLITAN W. AND S. BOARD, BOSTON, MASS.

	Cost per m. g. 1 ft.	M.G.D.	Lift, ft.	Cost coal.	Duty m. ft.-lbs.
Met. Sewer—					
Deer Island...	$0.117	58.6	10.9	$3.81@4.13	42.3
East Boston..	0.095	56.8	15.22	3.34@4.20	45.5
Charlestown .	0.150	35.3	8.95	3.83@3.90	55.5
Alewife Br'k..	0.50	3.56	12.84	4.23@4.73	18.6
Ward Street..	0.075	22.80	40.02	4.09@4.30	82.5
Met. Water—					
Chestnut Hill 1 and 2....	0.0492	3,77	119.67	$3.70	65.3
Chestnut H. 3	0.6342	0.62	129.88		104.7
Chestnut H. 4	0.0235	29.7	130.58		141.8
Chestnut Hill, low serv....	0.0365	50.2	49.08	3.46	107.75
Spot Pond....	0.0378	7.55	130.92	4.29	126.86

Repairs had been of high annual cost. On the high pressure engines particularly the cost of upkeeping on the auxiliary apparatus is very high. In justice to this type of engine, however, it must be said that foreign make of this same engine has proven much more reliable and less costly as regards maintenance. The truth of the matter seems to lie in the fact that the foreign-built engines have been better built than American-built machines reported to be of the same general design. Material advancement has been made of late, however, all along the line as regards American manufacture of oil engines. As for the lower pressure type, the superintendent stated that there had been considerable difficulty in maintaining alignment. The engines seeemed to pound out of line and apparently were not heavy enough for the work required of them. In each battery of engines one or more units was out of commission for repairs.

TABLE III.—OIL ENGINES—MILLION FOOT-POUND DUTY PER GALLON OF OIL.

Pints per HP-hr.	Lbs. per HP.-hr.	Gals. per HP.-hr.	Mill. ft.-lbs. per gal., 100 pct. eff.	Efficiency of pumps. Million Foot-Pounds Duty per Gallon of Oil. Efficiency of pumps													
				30 pct.	35 pct.	40 pct.	45 pct.	50 pct.	55 pct.	60 pct.	65 pct.	70 pct.	75 pct.	80 pct.	85 pct.	90 pct.	95 pct.
0.328	0.3	0.0412	48.0	14.4	16.8	19.2	21.6	24.0	26.4	28.8	31.2	33.6	36.0	38.4	40.8	43.2	45.7
0.438	0.4	0.0548	36.2	10.8	12.6	14.4	16.2	18.1	19.9	21.7	23.5	25.3	27.2	28.9	30.7	32.5	34.3
0.548	0.5	0.0685	28.8	8.65	10.1	11.5	13.0	14.4	15.9	17.3	18.7	20.2	21.7	23.2	24.55	26.0	27.45
0.650	0.6	0.0812	24.3	7.3	8.5	9.75	10.9	12.2	13.4	14.6	15.85	17.05	18.3	19.5	20.7	21.95	23.2
0.765	0.7	0.0959	20.65	6.2	7.25	8.25	9.3	10.3	11.35	12.4	13.45	14.45	15.5	16.5	17.5	18.6	19.6
0.875	0.8	0.1095	18.10	5.43	6.35	7.25	8.15	9.05	9.95	10.85	11.75	12.65	13.55	14.5	15.35	16.3	17.2
0.985	0.9	0.1235	16.00	4.8	5.6	6.4	7.2	8.0	8.8	9.6	10.4	11.2	12.0	12.8	13.6	14.4	15.2
1.095	1.0	0.137	14.50	4.35	5.08	5.8	6.52	7.25	7.96	8.7	9.43	10.15	10.9	11.6	12.3	13.05	13.75
...	1.2	0.165	12.10	3.63	4.23	4.85	5.45	6.05	6.65	7.25	7.85	8.45	....	....	....	....	....

Plant No. 6.—Information on this plant was obtained from one of the engineers operating ie plant for the bondholders. This was of ie high pressure type. and, perhaps, in part ecause of its remote location, has high maintenance charges. Depreciation is charged off ere on a 15-year basis and repairs amount to bout 8 per cent to 10 per cent annually. The pe of oil engine used here is practically obolete and both repair and depreciation harges unnecessarily excessive. It is said iat, in spite of the high charges for maintenance, crude oil as a motive power is cheaper ian steam for this location and, if the writer f this paper is correctly informed, additions re soon to be made to the oil engine plant.

Plant No. 7.—This plant consists of two staons, one with two engines and the other ith three. These are operated with producer as, one having a compression producer and ne a suction producer. The writer undertands that the cost of repairs for one plant as been comparatively small except that one ngine has recently broken its shaft. On the ther plant, however, the cost of repairs has een high and either one or the other of the vo units has practically always been out of ommission for repairs.

To sum up briefly, it is conceded by most uthorities on the subject that oil engine contruction in this country has previously been o light. It is also common knowledge that marked improvement has been made in reent years and that there is strong evidence f a determination on the part of the manuacturers to still further improve their output, especially with regard to strengthening eak parts.

One of the features of operation with which wners have had to contend has been the rejudice of steam engineers and firemen gainst oil engines. The writer believes it est not to employ steam engineers or fireien on explosion engines, but to train other ien for the work. Because of the constantly rowing automobile industry and the rapidly icreasing number of men who understand xplosion engines, it should be a simple matr to find men almost anywhere who can be eadily trained to operate the crude oil enine. The writer has seen explosion engines ackfired purposely by operators who held lienses as steam engineers.

As for the actual cost of maintenance and epairs, while it seems to be generally coneded among disinterested persons that the ost of repairs today need not be more than rom 2 per cent annually for the best Americ an engine to 4 per cent annually for some hich perhaps might not be termed first-class, or our study we assumed 10 per cent on the ne-fifth part of the installation of five engines hich was to constitute the plant.

COST OF PLANT.

The first cost of oil engines varies from $60 $80 per horsepower, to which must be add d the cost of the pump. The fuel consump on may be assumed to vary about as stated Table I.

The fuel consumption varies, of course, ith the make, etc. Add $0.0004 per HP.-hour or cost of lubricating oil and waste. The ost of crude oil at present is about 4 cts. per allon.

The first cost of steam engines and pumps about $50 per horsepower for the size under onsideration. To this, of course, must be dded the cost of steam boilers and the necesary auxiliaries and the steam piping.

The first cost of electric motors and pumps , of course, much less than that of either il engines or steam power plants.

The following statement showing comparave annual operating costs is a summary of ie studies made:

OMPARATIVE ANNUAL OPERATING COSTS—POWER, INTEREST, DEPRECIATION REPAIRS, ETC.—SEWAGE PUMPING—LIFT, 19.5 FT.

	1915.	1935.
v. daily dry weather flow, M.G.D.	10.35	18.3
v. daily dry weather flow, gals. per min.	7,200	12,700
heor. water end, HP.	35.5	62.5
P. at 60 pct. efficiency	59.2	104.2
W. req. at 51 pct. comb. eff. (85 pct. mot)	52.0	92.0

Average increase in above quantities to proide for storm water pumpage, 9 pct., based on experience of City of Providence, R. I., corroborated by Met. Sewer Reports:

	1915.	1935.
KW.-hrs. per yr., dry weath.	457,000	805,000
KW.-hrs. per yr., incl. storms	498,000	877,500
M ft. G. pumpage, dry weath. only	74,000	131,000
M ft. G. p'page, D. W., incl. storms	81,000	143,000

OIL ENGINES.

5 75-HP. engines at $4,500	$22,500
5 pumps at $1,300	6,800
	$29,300

Cost oil, 4 cts. gal—

$$\frac{\$0.04}{7.3} = \$0.0055 \text{ per lb.}$$

$$\frac{\$0.0055 \times 1.12 \text{ lbs.} \times 4.2}{0.60 \text{ (eff.)}} = \$0.043 \text{ per M. G. 1 ft. high.}$$

1.12 lb. per HP.-hr. is guarantee on ¾ load.

0.55 lb. per HP. hr. is Andover experience water works under uniform full load and high head.

1 lb. per KW.-hr. is exp. of electric plant = 0.75 lb. per HP-hr., varying load—but high pressure Diesel engines.

Labor cost—

1 engineer at $4.50	$ 4.50
3 engineers at 4.00	12.00
Per diem	$16.50

3 shifts and 1 extra man—7-day service.

$$\frac{16.50 \times 365}{81,000} = \$0.0745\text{—labor cost to raise 1 m. gals. 1 ft.}$$

Depreciation, interest, repairs—

		Total.
Life oil engine, 15 yrs.; int. 4 pct.; rep., 2 pct.; dep., 0.056		0.116
Life pump, 15 yrs.; int., 4 pct.; rep., 2 pct.; dep., 0.056		0.116
Buildings, assumed life, 40 yrs.; repairs, ¼ pct.		
63 × 80 × 25...$12,600	Int.	0.040
Foundations ... 15,000	Dep.	0.015
	Rep.	0.0025
$27,600		0.0575

2 pct. repairs on entire plant equivalent to about 10 pct. repairs on the one-fifth part of plant in constant use.

Labor cost to raise 1 m. gals. 1 ft.	$0.0745
Cost waste, etc., 1 m. gals. 1 ft. (incl. lub. oil)	0.008
Fuel cost to raise 1 m. gals. 1 ft	0.043
Total cost to raise 1 m. gals. 1 ft. (except int., dep., rep.)	$0.1255

Int., dep., rep. machinery, 29,300 × 0.116	$ 3,420
Labor and fuel cost, 81,000 × 0.1255	10,150
Int., dep., rep. building, 27,600 × 0.0575	1,590
Total annual cost, year 1915	$15,160

	1915.	1935.
Int., dep., rep. buildings	$ 1,590	$ 1,590
Int., dep., rep. machinery	3,420	3,420
Labor cost	6,000	6,000
Fuel cost	4,150	7,200
	$15,160	$18,210

STEAM ENGINES.

6 60-HP. at $3,000	$18,000
3 147-HP. boilers	4,024
Piping	3,600
Feed water heater and condensers	750
	$26,374

Cost coal—

Assume 40 m. ft.-lbs. duty per 100 lbs. coal = 20.8 lbs. coal per m. gals. 1 ft. high at $4 ton = $0.0416 per m. gals. 1 ft. high.

Fuel consumption based on experience Met. Sewer, Boston, and corroborated by many others, for varying load—storm water demands.

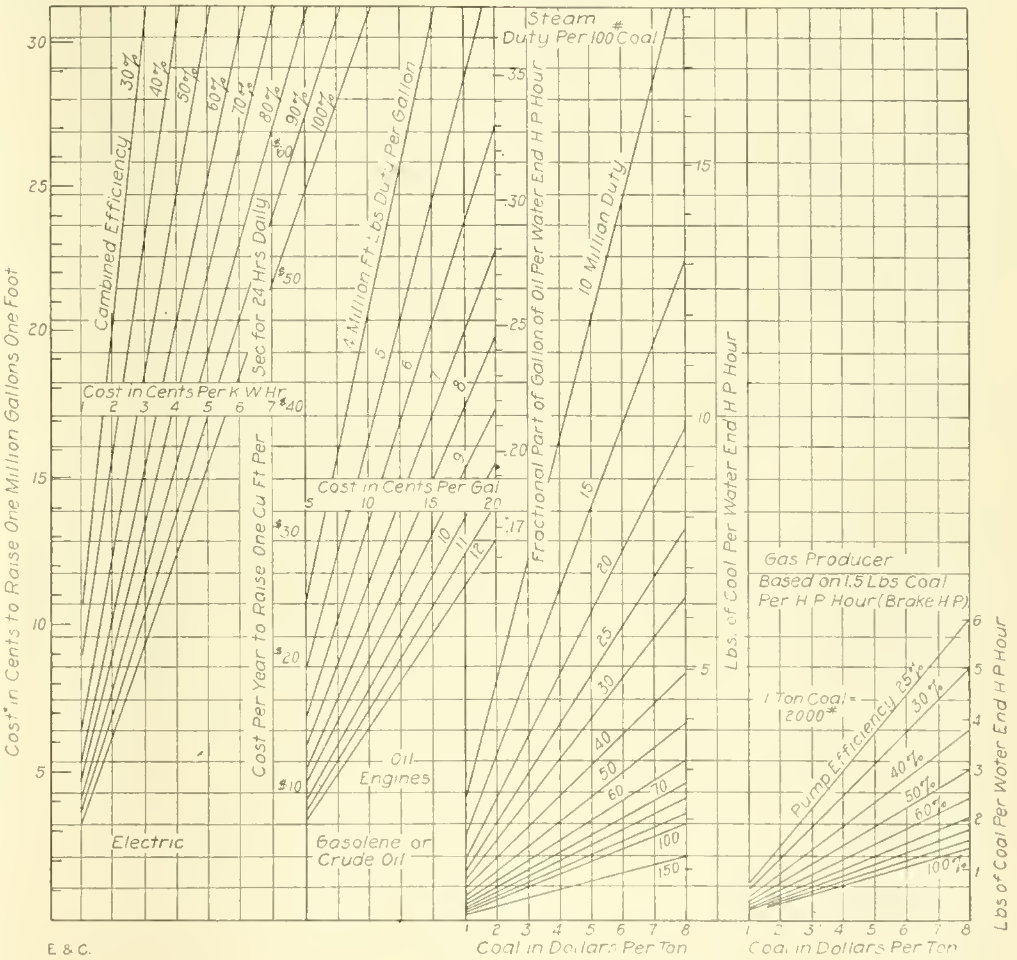


Fig. 2. Comparative Fuel Costs of Raising 1,000,000 Gals. 1 Ft. High, or of Raising 1 Cu. Ft. of Water Per Second 1 Ft. High for a Year, With Electric, Oil, Steam or Producer Gas Motive Power.

Labor cost—

1 engineer at $4.50	$ 4.50
3 engineers at $4.00	12.00
4 firemen at $2.75	11.00
3 shifts, 7-day service (1 extra man)	$27.50

$$\frac{\$27.50 \times 365}{81,000} = \$0.124 \text{ per m. gals. 1 ft. high.}$$

Depreciation, interest, repairs, etc.—

	Int.	Rep.	Dep.	Total.
Engine, life 20 yrs.	0.04	0.005	0.039	0.084
Boilers, life 15 yrs.	0.04	0.010	0.056	0.106
Piping, life 25 yrs.	0.04	0.005	0.029	0.074
Cond'sers, life 15 yrs.	0.04	0.005	0.056	0.101

Building—Assume 40 yrs. life and rep. at ¼ pct.

Engine room and pump, 63 × 80 × 25	$12,600
Boiler room, 63 × 49 × 25	7,700
Foundations	15,000
	$35,300

Labor cost to raise 1 m. gals. 1 ft.	$0.124
Oil, waste, etc., to raise 1 m. gals. 1 ft.	0.008
Cost coal to raise 1 m. gals. 1 ft.	0.0416
Total cost to raise 1 m. gals. 1 ft.	$0.1736

Int., dep., rep. engines and pump, $18,000 × 0.84	$ 1,510
Int., dep., rep. boilers, $4,024 × 0.106	427
Int., dep., rep. piping, $3,600 × 0.074	267

Int., dep., rep. accessories, $750 × 0.106..	80
Int., dep., rep. machinery	$ 2,284
Labor and fuel cost, 8,100 × .1736	$14,020
Int., dep., rep. building, 35,300 × .0575	2,030
Yearly cost, 1915	$18,334

	1915.	1935.
Int., dep., rep. buildings	$ 2,030	$ 2,030
Int., dep., rep. machinery	2,284	2,284
Labor cost	10,000	10,000
Fuel cost	4,020	7,000
	$18,334	$21,314

A point in favor of the steam engine is that as the quantity increases the duty per 100 lbs of coal will improve. This will apply to the storm water pumped, vide experience at Charlestown metropolitan sewerage system.

TABLE IV. STEAM ENGINES—MILLION FOOT-POUNDS DUTY PER 100 LBS. OF COAL.

Lbs. coal per brake HP.-hr.	Duty, M. ft. lbs., theoretically perfect pump. 100 pct. eff.	Million foot-pounds of duty per 100 pounds of coal 30 pct.	40 pct.	50 pct.	60 pct.	70 pct.	80 pct.	90 pct.	95 pct.
0.4	495.0	149.0	198.0	.....	.....	.....	.....	.....	.....
0.6	330.0	99.0	132.0	165.0	198.0	.....	.....	.....	.....
0.8	247.0	74.0	98.5	123.0	148.0	173.0	197.0	.....	.....
1.0	198.0	59.5	79.0	99.9	119.0	139.0	158.0	178.0	188.0
1.2	165.0	49.5	66.0	82.5	99.0	115.0	132.0	149.0	157.0
1.4	141.0	42.3	56.5	70.5	84.5	99.0	113.0	127.0	134.0
1.6	124.0	37.2	49.5	62.0	74.5	86.8	99.0	111.0	118.0
1.8	110.0	33.0	44.0	55.0	66.0	77.0	88.0	99.0	.....
2.0	99.0	29.7	39.6	49.5	59.2	69.1	79.0	89.0	.....
2.2	90.0	27.0	36.0	45.0	54.0	63.0	72.0	81.0	.....
2.4	82.5	24.7	32.9	41.2	49.5	57.5	66.0	74.0	.....
2.6	76.0	22.8	30.3	38.0	45.6	53.2	60.8	68.5	.....
2.8	70.5	21.2	28.2	35.3	42.3	49.3	56.3	63.5	.....
3.0	66.0	19.8	26.3	33.0	39.6	46.2	52.8	59.5	.....
3.5	56.6	17.0	22.7	28.3	33.9	39.6	45.3	50.9	.....
4.0	49.5	14.9	19.8	24.7	29.7	34.6	39.6	44.5	.....
4.5	44.0	13.2	17.6	22.0	26.3	30.8	35.2	39.6	.....
5.0	39.6	11.9	15.85	19.8	23.7	27.7	31.7	35.7	.....
6.0	33.0	9.9	13.2	16.5	19.8	23.2	26.3	29.7	.....
7.0	28.3	.	11.3	14.1	17.0	19.8	22.7	25.5	.....
8.0	24.7		9.9	12.3	14.9	17.3	19.8	22.2	.....
9.0	22.0	.	....	11.0	13.2	15.4	17.6	19.8	.....
10.0	19.8		...	9.9	11.9	13.9	15.8	17.8	.....

ELECTRIC MOTORS.

75-HP. motors at $1,500	$ 7,500
Switchboards	2,500
Wiring	1,500
	$11,500
pumps at $1,050	5,250
	$16,750

Cost electricity—

Pump, 60 pct. efficient.

Motor, 85 pct. efficient.

Combined efficiency = 0.51.

1 m. g. 1 ft. high = 4.21 HP.-hrs., theoretically = 6.16 KW.-hrs. at 51 pct. combined eff.

1¾ cts. KW.-hr. = $0.108 m. g. ft.

1 m. g. 1 ft. high = 4.21 HP.-hrs., theoretically.

1 chief operator at $4.50	$ 4.50
3 operators at $4	12.00
	$16.50

$\frac{\$16.50 \times 365}{81{,}000}$ = $0.0745 cost of labor to raise 1 m. g. 1 ft.

Depreciation, interest, repairs—

	Int.	Rep.	Dep.	Total.
Motors, life 25 yrs.	0.01	0.005	0.029	0.074
Pumps, life 15 yrs.	0.01	0.020	0.056	0.116

Building—Assume life 40 yrs. Repairs, ¼ pct.

63 × 80 × 25	$12,600
Foundations	10,000
	$22,600

Cost electric current, 1 m. g. 1 ft. high	$0.108
Cost waste, lub. oil, etc., 1 m. g. 1 ft. high	0.008
Labor cost to raise 1 m. g. 1 ft. high	0.0745
Total cost to raise 1 m. g. 1 ft. high	$0.1905
Dep., int., rep. motors, $7,500 at .074	$ 555
Dep., int., rep. switchb. and wire, $4,000 at .074	296

TABLE V.—COMPARATIVE OPERATING ECONOMY WITH OIL, STEAM AND ELECTRIC MOTIVE POWER FOR 1915 AND 1935.

Year 1915

	Oil.	Steam.	Electric
[illegible]	2	1	3
Labor	1	2	1
Int., dep., machinery	3	2	1
Int., dep., buildings	2	3	1

Year 1935

	Oil	Steam	Electric
[illegible]	2	1	3
Labor	1	2	1
Int., dep., machinery	3	2	1
Int., dep., buildings	2	3	1

Dep., int., rep. pumps, $5,250 at .116	610
[illegible] cost (incl. labor, elec., waste), [illegible]	15,400
Int., dep., rep. building, $22,600 × .0575	1,300
Yearly cost	$18,151

	1915.	1935.
Int., dep., rep. motors	$ 555	$ 555
Int., dep., rep. switchb., wire, etc.	296	296
Int., dep., rep. pumps	610	610
Labor cost	6,000	6,000
Cost current, waste, etc.	9,400	16,300
Int., dep., rep. buildings	1,300	1,300
	$18,151	$25,061

The present average dry weather flow for 1915 was estimated at 10,350,000 gals. daily, while that for 20 years in the future was estimated from past census returns compared with the per capita water consumption in years past. The lift was to be 19.5 ft. and the water end or theoretic horsepower was to be 35.5 HP. in 1915. From a study of the experience of Providence, R. I., as well as the records of the pumping of the Metropolitan Sewerage Works of Boston, Mass. (see Table II), it was estimated that the total pumpage for one year would be increased about 9 per cent on account of storms.

OIL ENGINES.

It was deemed necessary to provide for a storm capacity of at least four times the required average daily dry weather flow and to provide another unit for repairs of oil engines. It was estimated that at least one oil engine might be out of commission for repairs when most needed, hence five oil engines were provided for in the estimate.

The fuel consumption estimated was 1.12 lbs. per B. HP.-hour, as a guarantee can be had from one or more of the manufacturers of the better class of oil engines at that figure for three-quarter load. Data pertaining to oil engine operation are given in Table III.

The Andover Pumping Plant for pumping the water supply of the town of Andover, Mass., consumes but 0.55 lb. per HP.-hour under a uniform load and high head. As the load on the plant under consideration would be extremely varying and the units would be operating under partial load most of the time, the guarantee for three-quarter load appeared a most conservative estimate. It was found that the low pressure engine with its consequent lower duty per gallon of fuel was the cheapest installation for this plant. This is due to the fact that the full capacity of the plant of four engines is required for but about 30 days of the year. This means in effect that three of the engines are idle eleven months of the year and the fifth one is idle all the time as a spare engine for use during repairs.

The life of the oil engines was assumed at 15 years and repair charges of 2 per cent on the entire plant allowed. This is equivalent as before stated, to a charge of 10 per cent on the one unit in constant use, which seems a very high allowance.

Because of the laws of the state of Massachusetts with respect to laborers and engineers and firemen working no more than eight hours a day, six days a week, it is necessary to provide four men to operate three shifts of eight hours per day. In the case of Salem, Mass., pumping station, the four men have arranged to work 6 hours each day and occasionally, when one wishes a day off, the other three work eight hours each for the day.

The required size of building was worked out in each case and a life of 40 years with ¼ per cent repairs annually was assumed.

STEAM ENGINE PLANT.

It was deemed best, in the case of the steam engine plant as proposed, to divide the plant into six pumping units and three boilers.

A "station" or "plant" duty of 40,000,000 ft. lbs. per 100 lbs. of coal was assumed as what might be expected from a plant of this size under the extremely varying load which is to be expected from day to day, in fact from hour to hour.

Here again it was necessary to provide for four men to operate three shifts per day. It is, of course, obvious that a steam plant will require a fireman in addition to the engineer and this means, under our Massachusetts State Labor Law, four men to operate three shifts. A life of 20 years for the engine, 15 years for the boilers, 25 years for the piping and 15 years for the condensers was assumed as a fair basis for comparison with the other plants. A building was laid out roughly and its cost determined approximately, and a life of 40 years and an annual repair charge of ¼ per cent assumed as for the other plants. Data pertaining to steam engine operation are given in Table IV.

ELECTRIC PUMPING.

Here again it was necessary to allow 4 men per day to provide for three shifts per day under the labor laws of the state. A life of 25 years was assumed for the motors and 15 years for the pumps.

All the *assumptions* made are decidedly unfavorable to the oil engine; that is, as a matter of practice, the fuel consumption would probably be reduced from 30 per cent to 40 per cent. The depreciation charge is too high. The repair charge of 2 per cent on the entire plat or 10 per cent on the part which operates continually is probably high. In spite of all the foregoing the total tabulated comparative result given above and shown graphically in Fig. 1, apparently favors the oil engine.

Much of the opposition against the oil engine perhaps comes from the careful and orderly mechanic who dislikes to see a machine out of commission for any reason whatever, even although the books show that the saving in fuel consequent to their use is enough to pay for the necessary repairs several times over. Not many care to have charge of a machine which is constantly needing repairs, but if the large repair bill plus the small fuel and small labor bill of one machine is much cheaper than the small repair bill plus the small fuel bill and doubled labor bill of the second machine, the investor will undoubtedly demand the first for highest returns on the investment.

Table VI is a table of constants and equivalents by which the work was computed.

TABLE VI.—TABLE OF EQUIVALENTS AND CONSTANTS.

1 C. F. S. 1 ft. high = 0.1136 HP. per hour.

1 m. g. 1 ft. high = 4.21 HP.-hours.

1 C. F. S. = 0.646 m. g. daily = 236 m. g. yearly = 995 HP.-hours.

86,400 seconds in one day.

8,760 hours in one year.

100 million duty requires 8⅓ pounds of coal per 1,000,000 gallons 1 ft. high = 1.98 lbs. coal per HP.-hour.

	Elec., 100 pct. eff.		Oil, 10 m. duty.		Steam, 100 m. duty.	
Cost to raise 1 m. g. 1 ft.	$ 0.0315	x	$0.0083	y	$0.004167	z
Cost to raise 1 C. F. S. 1 ft. high 1 yr.	7.40	x	1.971	y	0.985	z
Cost per w. h. p.-year	65.40	x	17.33	y	8.70	z
Cost per w. h. p.-hour	0.00746	x	0.00198	y	0.00985	z

Where x = cost of current in cents per K.W.-hr.; y = cost of oil per gallon in cts., and z = cost of coal per ton in dollars.

The comparison of 1935 is still less favorable to the other kinds of motive power (steam and electric), particularly so in the case of electricity. Of course, one might say that no one knows what the price of oil will be 20 years hence, though it will probably be higher. Neither can one predict the price of electricity. Certainly if some one succeeds in extracting it directly from the fire the price will drop considerably. In matters of engineering, however, it is obviously unsafe to make provisions for visionary possibilities.

Figure 1 shows graphically the relation between the annual costs for both years 1915 and 1935, at the beginning and at the end of the period under consideration. The relative yearly costs of fuel, labor and interest and repairs and depreciation on machinery and buildings are very clearly set forth.

Figure 2 shows the comparative fuel costs of raising 1,000,000 gals. 1 ft. high, or of raising 1 cu. ft. of water per second 1 ft. high for a year, either by electricity, oil, steam or producer gas. This diagram of the cost of fuel should, of course, be studied in connection with a study into the question of the cost of attendance. For instance, neither electric pumping nor (in all probability) oil pumping will require a man or laborer corresponding with the fireman required for a steam plant or a gas producer plant. The size of a steam plant which one man can handle; that is, do both firing and operate the pumping engine, is a point which should be investigated for small plants before selecting the motive power. One experienced operating and designing mechanical engineer has stated that one man can both fire and operate plants up to 100 HP. The introduction of steam turbine driven units for pumping water should raise the figure representing the size of the plant which one man can operate.

The itemized annual cast of operation estimate, previously described and stated, is a comparison of the yearly operating costs of pumping the sewage of the city of Lynn of 100,000 inhabitants either by oil, steam or electric motive power. It was from this study that Fig. 1 was drawn. The relative advantages of each motive power are still further indicated in Table V by rank with respect to economy.

Roads and Streets

Methods and Costs of Maintaining Cement Concrete Roads With Bituminous Surfaces.

Contributed by Herbert C. Poore, East Braintree, Mass.

The troubles arising from the cracking of concrete roads have led to the use of a thin bituminous surface as a method of maintenance. Such a treatment produces a noiseless, resilient, gritty surface, depending upon the concrete beneath for strength. In cities where unsightly cracks would invite public criticism and would quickly increase in width under iron tire vehicles, the film treatment aids in justifying the use of concrete pavements. Owing to its lack of resistance to abrasion concrete rapidly chamfers at all contraction cracks unless they are at once filled with a suitable material, those made of bituminous materials being the kinds most generally used.

Small differences of elevation in a concrete surface and elevations due to manhole covers and car tracks, often cause unevenness in the best concrete without a bituminous surface. A coat of bitumen, through its tendency to flow to the low spots, evens up such irregularities, and if the bitumen is of the right consistency, it will produce a smoother riding surface than can ever be obtained on a concrete surface itself. With traffic and warm weather, the proper bitumen will completely work into and seal the smallest cracks. It will even sink into the concrete itself to a depth of 1/16 in.

Very little study has been given to the maintenance of bituminous films used as a protecting coating over concrete roads and few published data are available. During the past two seasons the treatment of some miles of both new and old concrete in New England, some of which had been previously treated with bitumen, has afforded a good field of observation.

BITUMINOUS SURFACES ON NEW CONCRETE ROADS

The new road consisted of a 6-in. 1-2-4 cement concrete slab laid with a machine mixer having a distributor arm with a bucket. Three-quarter inch contraction joints every 25 ft., filled with coal tar pitch, were provided. These were poured with the hot pitch immediately after removing the separating strips, in order to prevent dirt from being carried into the joint. Practically all the work was open to travel before the application of the bituminous surface, necessitating a thorough cleaning of the concrete by hand. The bitumen, in this case a refined tar, was heated in tank cars at the nearest siding, pumped into a steam pressure sprayer and applied to the concrete in two coats of ¼ gal. each per square yard. As the distributor sprayed a strip 8 ft. wide, much care was necessary to avoid a wide and uneven overlapping of the strips. The streak resulting is very noticeable over a hard surface with even a ¼-gal. per square yard application.

Half-inch clean peastone was spread over the first coat at the rate of 1 cu. yd. to 200 sq. yds., and the surface well rolled. The second application of bitumen was then immediately made in a similar manner before any traffic was allowed over the first coat. This was covered with peastone and a further light blotting coat of stone dust or sand, using a total quantity of covering over the second coat of 1 cu. yd. to 200 sq. yds. of surface.

As the width of concrete was either 10 to 12 ft., macadam shoulders were provided. These were in some instances treated with bitumen for a portion of their width, in the same manner as the concrete. Defects in the bituminous coat soon appeared on these shoulders under traffic, due to the difference in hardness of the concrete and the macadam.

The following specifications were allowed on the 1913 work:

SPECIFICATIONS FOR APPLICATION OF BITUMINOUS SURFACE OVER CEMENT CONCRETE BASE.

Protection After Laying.—After the concrete is laid and until it has thoroughly set, it shall be protected from the sun by a canvas or other suitable covering in a manner to be prescribed by the engineer. When in the judgment of the engineer the concrete is sufficiently hard to warrant, this covering shall be removed and the concrete covered with a layer of sand or gravel or other material to be approved by the engineer to about 1 in. in depth. The surface shall be sprinkled or kept damp for at least seven days, to prevent the surface of the concrete from drying out while setting. The sand, or other material, shall then be removed from the surface and disposed of in a manner satisfactory to the engineer.

Wearing Surface.—After the concrete is thoroughly dry and set, it shall be carefully swept and cleaned. After the seven days have elapsed and when the surface is absolutely dry, an even coating of bituminous material, heated to not less than 200° F., shall be spread over the surface in a uniform layer by means of a suitable pressure spraying device. Approximately ½ gal. per square yard shall be used. This shall be spread on the surface in two layers of not less than ¼ gal. per square yard in each layer. As soon as the first application of bituminous material is made, it shall be covered with a uniform layer of peastone and rolled with a roller approved by the engineer. On the surface thus prepared a second coat of bituminous material, of not less than ¼ gal. per square yard, is to be made. The surface shall then be covered with peastone and rolled. Approximately 1 cu. yd. of peastone shall be used to 200 sq. yds. of surface in each layer. A sufficient amount of peastone shall be left on the surface to protect the road while setting up.

CONCLUSIONS

In carrying on this work several important points were developed, as follows:

(1) The finished thickness of the film should be as small as possible, consistent with a proper application of the bitumen and the grit. The advantage of a thin film is that when worn spots appear the slight difference in elevation between the base and the coated concrete is not noticeable under travel. With the development of a small number of weak spots, it is not wise to at once make an immediate retreatment of the entire surface, as some portions would then have a double coat. A careful hand mopping of these spots with small quantities of bitumen will defer the day of general recoating.

(2) The application of two ¼-gal. coats with grit between gives a much better mixture of mineral and bitumen than one application of ½ gal. The former produces a surface of well mixed bitumen and grit which shows little tendency to roll or move about on the concrete. This oftentimes happens with a ½-gal. coat in one application, notwithstanding a good bonding to the clean concrete surface.

(3) Much better results are obtained with power sprayers giving a uniform sheet of bitumen. Hand work, either with a single pressure nozzle or by hand pouring and sweeping does not yield a uniform coat. The unevenness produced at the start is generally aggravated by traffic.

A warm, dry concrete tends to absorb some of the lighter oils from the tar bitumen and the concrete is often discolored to the depth of 1/16 in. The application of a priming or filler coat of a thinner bituminous material has been tried with the added object of keeping the tar oils from being absorbed from the first ¼-gal. coat. It is hoped in this way not only to obtain a closer bond, but to lengthen the life of the coating. The priming coat has been made fluid enough to flow cold and has been applied in such small quantity that the concrete takes it up after a few hours and presents a dry, tight surface for the first ¼-gal. coat of bitumen to adhere to. This results in better adhesion, especially if the concrete is slightly dusty, since the thin priming coat penetrates the dust to the concrete beneath.

TABLE 1.—COST OF A TWO-COAT BITUMINOUS SURFACE ON A CONCRETE ROAD.

	Per sq. yd.
Tarvia "B" at $0.105 per gal. applied, $0.02 gal. per sq. yd.	$0.021
Tarvia "A" at $0.12 per gal. applied, $0.25 gal. per sq. yd.	0.030
Peastone at $2.30 per cu. yd., including spreading, 1 cu. yd. peastone per 100 sq. yds. 1 sq. yd.	0.023
Total cost per sq. yd.	$0.074

COST OF FILM TREATMENT.

In two cases the detailed cost data of making the original film treatment on 1-2-4 mixture concrete roads were as follows:

Case (1).—A concrete road was covered with 0.2 gal. of Tarvia "B" per square yard, followed with 0.25 gal. of Tarvia "A" per square yard and covered with peastone at the rate of 1 cu. yd. to 100 sq. yds. of surface. A pressure distribution was used in applying the two coats of Tarvia. The treatment cost as shown in Table 1.

No charge is included for apparatus which was loaned for the work. Ordinarily a slight increase in cost would be necessary for this item. The quantity of peastone required is variable, depending somewhat on the temperature prevailing during the work.

Case (2).—This treatment was applied to a concrete road and consisted of 0.5 gal. of Tarvia "A" per square yard covered with peastone. The cost is given in Table 11.

TABLE II.—COST OF A ONE-COAT BITUMINOUS SURFACE ON A CONCRETE ROAD.

	Per gal.
Tarvia at side track	$0.08
Heating in tank car	0.02
Hauling 4 miles	0.006
Applying with steam sprayer	0.005
Cost of Tarvia "A" per gal	$0.111
	Per sq. yd.
One-half gal. Tarvia "A" at $0.111	$0.055
Local peastone at $1.42 per cu. yd., 1 cu. yd. peastone per 100 sq. yds., 1 sq. yd.	0.0142
Total cost per sq. yd	$0.0692

No charge is included for apparatus which was loaned, as in Case 1. A slight increase in cost would ordinarily be incurred for this item. The amount of peastone needed varies with the temperature as stated in Case 1.

BITUMINOUS SURFACES AND REPAIRS ON OLD CONRETE ROADS.

As a means of repairing pot-holes in concrete roads, chiseling out the old concrete to a square shoulder and filling with new concrete has proven slow and expensive, and it leaves a poorly bonded edge which is a source of weakness to the surrounding pavement. For the most part, concrete road repairs have been made with bituminous materials, using either the penetration or the mixing method. If the work is properly done, a good bond to the adjoining concrete is secured which excludes all water; and if the mixture is properly graded and stiff enough, the bituminous patch can be made practically equal to the concrete in resistance to wear. Aesthetically white concrete spotted with black patches is objectionable, hence such a patched surface is usually given a bituminous film treatment.

Recoating a concrete surface after the first application has partially worn off presents many difficulties if the resulting surface is to be smooth and durable as in the beginning.

Two conditions usually obtain: (1) Where the travel is confined to a limited width of the entire road; and (2) Where worn spots are scattered over the whole surface. In the former, especially on streets having car tracks through the center, and a 9 to 15 ft. roadway on each side with considerable traffic, the film practically disappears after a season's wear, so that the following spring the concrete surface is practically bare and ready for a second application of two ¼-gal. coats. The retreatment should only be given on the traveled section, omitting the coating next to the curb, if 12 or 18 ins. in width of the old material remain.

If worn spots are scattered over the entire area of the roadway, they will be found to be the high points of the original concrete. The bitumen has flowed into the sags of the concrete surface, leaving the high spots with a thinner film than the rest. If such is the case, the original specification for a film may be followed, provided the old bitumen is firm and stiff enough to support a second coat. Such a surface is far more even after the second film has been applied than after the first treatment.

Up to the present time, many failures in bituminous surfaces have been due to poor workmanship and the wrong quantity or kind of bitumen. Over a smooth, broomed concrete surface, a film composed of two ¼-gal. coats of refined tar properly applied, will have so little noticeable unevenness as it wears off that a retreatment of one or two ¼-gal. coats will not add appreciably to the film, if continued indefinitely at proper intervals.

Where, however, ¼ gal. coats have been applied in a crude manner to dirty concrete, scaling results and noticeable holes appear in the film surface. The surface is then very difficult to restore, as the film in places is over ½ in. in thickness and breaks with sharp shoulders at the edges of the holes. As on macadam roads, heavy films or mats in this condition have been extremely hard to cope with, and have probably done much to discourage the method. Lighter materials acting as a solvent have often been found a satisfactory remedy. Scraping and rolling the surface in the presence of water after such an application has removed much unevenness. This would also apply to a bituminous film over concrete. Many concrete surfaces have, however, been retreated with no attention whatever given to the old material, resulting in an uneven surface.

New coats of the same or even a lighter film coat should not be added until at least a large part of the old has disappeared, and every precaution should be taken to secure an even surface, as much patching is always unsatisfactory. Old tar bitumens usually adhere strongly to concrete and cannot be removed by scraping. Uneven heavy films on macadam roads can be scarified and the macadam smoothed and rolled. This method is impossible on concrete. Far more care is necessary consequently to start with a smooth concrete surface and to keep it smooth by intelligent and careful subsequent treatment.

Methods and Cost of Transplanting Street and Boulevard Trees in Paris, France.

The raising, planting and upkeep of the trees which so beautify the streets and boulevards of Paris are under the direction of the prefecture of the Department of the Seine, in which Paris is situated. A special administrative department of the prefecture is in exclusive charge of this undertaking, which, in a city like Paris, assumes very large proportions.

A recent issue of the "Daily Consular and Trade reports," prepared by the Department of Commerce, contained a report by Vice-Consul General Lucien Memminger, Paris, concerning boulevard and street trees in Paris from which the information here given is taken:

On Jan. 1, 1912, the total number of trees on the streets, boulevards and squares of Paris was 86,686; the total number of men employed throughout the year in planting, transplanting, watering and removing dead trees was 120; and the total cost of maintaining these trees was estimated at $96,500.

On an average 1,100 to 1,200 new trees are planted every year to replace those which have died, been overthrown by carts, automobiles, etc., or which have been removed for other reasons.

THE PLANTATIONS.

The city of Paris has large and varied plantations just outside the fortifications, where flowers, exotic shrubs, and trees are raised. One hundred and thirteen hothouses for plants, exotic shrubs and flowers are heated from one central heating plant, distributing hot water through a 9½-mile system of pipes. The object of this central heating organization is not, as the head gardener explains, with a view to economy. In fact, such a system is far more costly than separate installations for each group of hothouses, but is thus arranged to avoid the existence of a series of unsightly chimneys which would destroy the artistic effect of the grounds, which are more or less a public resort and show place.

In the tree and hardy shrub section of the plantations, which are at some distance from the flower and exotic plant collection, are dwarf pyramid elms and chestnuts carefully grafted to produce a double flower and on which no chestnuts ever form. This feature of the chestnut is extremely valuable in cities, where mischievous school children would do much harm to trees bearing fruit or nuts by throwing sticks and stones to knock them down.

Here are grown the greater number of the trees to be planted in the streets of Paris, the average age of which on transplantation is stated at ten years.

METHODS OF TRANSPLANTING.

The transplanting of small trees is easily accomplished, while for tall and heavy trees the depot of the city of Paris, in the heart of the Bois de Boulogne, has varied machinery, including 15 tree conveyors of varying sizes and models, ranging from those capable of conveying a 50 or even 60 ft. giant down to those for handling a 10 or 12 year tree.

The conveyor most generally used is a 4-wheeled wagon about 6 ft. high provided with a windlass on either side. The wagon, the front, back and sides of which form a rectangular parallelogram, is backed toward the tree to be removed, and the rear beam is taken off, thus allowing an entrance for the tree into the center of the wagon. The back of the wagon is replaced and the tree then hoisted by windlasses until it is clear of the ground. After removal to the spot where it is to be replanted, the operation is reversed. A similar wagon is provided with a crane fixed in front, which allows the lowering of the tree which is being conveyed to permit its passing under bridges, telegraph wires, and other obstacles.

The cost of transplanting naturally varies according to the size of the tree and consequent difficulty of handling. It is stated, however, that the average cost of transplanting the 1,100 or 1,200 trees needed annually in the streets of Paris is approximately $9.65 per tree. This figure may seem high, but the great majority of these trees are 20 or more years old, and are required to replace those of the same species which have died, and in the aproximate cost given are included not only manual labor, but upkeep of horses and wagons for transplanting, replacing in many cases all the earth around the roots when the quality is found to be of the wrong kind, and installation of the special watering apparatus mentioned below. When new streets or boulevards are planted out the trees employed do not exceed ten years in age, but when an old tree dies everything is done to replace it by one of similar size to those already in line with it.

UPKEEP AND WATERING.

The usual proceeding when trees are transplanted is to place around, or rather above and below, its roots a vertical square of brick piping, the lower side of the square which passes either completely below or at least through the roots of the tree being about 23½ ins. below the surface of the earth. The piping, the diameter of which is usually 1½ ins., the length of each joint being from 12 to 15 ins., lets out the water at each joint, and thus thoroughly soaks the earth around the roots. Watering is carried on through two orifices at each upper corner of the square of piping.

This system, however perfect it may be for insuring the successful transplanting of a tree, is said to be far from satisfactory after the first few years. The head gardener of the city of Paris states that, as a rule, after three or four years the piping becomes dislocated either through the spread of the roots or the sinking of the earth around the tree, or both. It is still used, however, on account of the excellent results it gives when the tree is first transplanted.

When a tree has been in the ground for two or three years, the earth around it is cleared out in the form of a basin, the greatest depth being from 5 to 6 ins. and the diameter of the basin 6 ft. Over this basin, and, consequently, around the tree, is placed a grating, through which the tree is watered, with an ordinary hose pipe, the basin being filled up to the level of the surrounding pavement. This grating-covered basin thus allows the necessary amount of air to penetrate to the roots.

Drainage Work in Pecos Valley, New Mexico.—Engineering work is underway for three drainage districts in the Pecos Valley, New Mexico. The preliminary work is being brought to a close and estimates, etc., will be completed probably in about 60 days. The approximate area of the three districts is 50,000 acres, and the cost of the system will be about $15 to $20 per acre. There will be no open ditches, the system to be installed will be of burnt clay tile. Eccles & Vauchelet, Roswell, N. Mex., are engineers for the three districts.

THE PANAMA CANAL

Outline History of Projects for a Canal Across the Isthmus of Darien With a Particular Account of the Structures for and the Methods and Costs of Constructing, the Canal at Panama.

(Staff Article)

INTRODUCTION

All parts of the Panama Canal are sufficiently completed for the passage of ships with the exception of Culebra Cut. There yet remain much finishing work and construction work on permanent terminals and living quarters and there remains considerable channel widening to be done on the sea level sections; none of this remaining work prevents the use of the canal. The remaining excavation at Culebra, due to slides, still prevents canal operation and, since no one can say with certainty what may be the future slide development, it cannot be told when this obstacle will be finally removed. The work remaining to be done at Culebra and in widening the sea level channels means also only a continuation of methods of work followed from the beginning and the volume of the remaining work cannot be more than a small percentage of the volume already completed. For these reasons an outline of the history of the Panama Canal, a discription of the canal structures and a statement of construction methods and costs can be made nearly as definative now as a few months hence when the great waterway, it is anticipated, will be opened for traffic.

The following article gives first an outline chronological account of the studies and plans which finally culminated in the plan that has been carried virtually to completion. It describes all the main canal structures, the dams, the locks, the Culebra cut, and the sea-level channels, and it also describes for each of these structures the plant and methods employed for construction. Construction costs are given for all the main units of the canal work. The work of sanitation is outlined, the cost-keeping system is explained and the terminals, lighting system, shop system, power plants, construction plant equipment and costs are scheduled and discribed.

The information presented has beeen compiled chiefly from the reports of the Isthmian Canal Commission and amplified by reference to articles in ENGINEERING AND CONTRACTING, "Engineering News," "Engineering Record," "Transactions American Society of Civil Engineers, "Journal of the Association of Engineering Societies," and other publications. Most of the illustrations have been redrawn from the reports of the Isthmian Canal Commission and others have been reproduced from the "Engineering News" and the "Engineering Record," after being checked against the official plans for accuracy.

With so voluminous a record from which to draw it has been necessary to condense many parts to the very briefest of statements and to omit some features of construction whose mention was not vital to a clear picture of the work. It has also been the purpose to give as many statistics as would make evident the cost of construction, the performance of plant, the progress of construction and the volumes of work involved. This plan does not obstruct the engineer's view of the immensity of the work as a whole and it gives him the data which will be most useful to him in determining the construction efficiency of the Panama Canal labor forces

History of Projects for a Trans-Isthmian Canal

There are three periods of trans-isthmian canal history: The period of projects; the period of French construction, and the period of American construction. A statement follows of the principal events of each period. For this outline facts have been taken from various sources but the chronology is that of Mr. Joseph Bucklin Bishop, Secretary Isthmian Canal Commission, in his interesting work "The Panama Gateway"

Period of Projects.

The seeking for a waterway across the Isthmus of Darien is as old as Columbus. The great discoverer and the navigators to the new world who followed him searched every inlet of the Isthmian coast for a passage for ships. In 1513 Balboa crossed Darien by boat and trail and discovered the Pacific Ocean. Four years later Panama was settled and in 1519, Nombre de Dios on the Atlantic was founded.

To connect Panama and Nombre de Dios there was built in 1519 a paved trail. In 1535 a boat line was established from Cruces, where this trail touched the Chagres River, to the mouth of the Chagres. Meanwhile, in 1532, Lake Nicaragua was discovered and trafic by boat and trail established a new trans-isthmian crossing. Charles V. was then king of Spain, and if history speaks truly, he seems first of all to have conceived the idea of a trans-isthmian canal He decreed in 1534 that a survey be made from Cruces to Panama to determine the practicability of connecting the Chagres by artificial waterway with the Pacific. His answer from his new world possessions was that such a work was beyond the effort of man and the treasuries of kings. In 1567 Philip II, the son of Charles V., following the precedent set by his father, ordered surveys to be made for a canal crossing at Nicaragua. The report he received was scarcely more encouraging than that which, a third of a century before, came to Charles V. from Panama.

In 1597 the Atlantic terminal of the Panama route was changed from Nombre de Dios to Puerto Bello and new trails were constructed between Panama and Puerto Bello. These trails and also the old road to Cruces are indicated on the map, Fig. 2 For two centuries Spanish trans-isthmian traffic made no attempt of record to secure better routes than these river and land trails at Panama and Nicaragua. Want of enterprise was not wholly the reason for this. England controlled the seas and Spain did not care to provide a trans-isthmian waterway for the gentlemen buccaneers under Morgan, Drake and Frobisher. In 1814, however, the Spanish Cortes issued a decree providing for a trans-isthmian canal and for a company to build it. A few years later, 1819, Central America achieved its independence from Spain and by 1823 Spain had lost all its Central and South American colonies. Spanish connection with trans-isthmian canal enterprises ceased.

The new Isthmian republic began almost at once to invite outside interests to consider trans-isthmian canal construction. In 1825 Aaron H. Palmer, a New York merchant, began negotiations with the Republic of Central America with the view of building a canal at Nicaragua, and he finally secured a concession. Palmer enlisted the moral support of the United States government for his plans but after working on them about twelve months he abandoned his project. In 1825 also, an English company sent an engineer, John Bailey, to survey the isthmus for a canal at Panama; this company did not get so far as to obtain a concession. In 1830 the Republic of Central America trafficked with Netherlands merchants for a canal but without material results. It then, in 1835, turned to the United States and, following a favorable resolution of Congress, Charles Biddle was sent to Nicaragua to examine routes for a canal; the result was a report to Congress by President Jackson in 1837 that the enterprise "was inexpedient." In 1839, when Van Buren was President, John L. Stephens was sent to the Isthmus to report on routes for a canal; he recommended the Nicaragua route and estimated the cost of a canal there at $25,000,000, but advised against its construction.

In 1831, the Republic of Colombia, composed of New Granada, Venezuela and Ecuador, separated into its component parts. The Panama route was in New Granada and in 1838 this country granted concession to a French company to construct roads, railways or canals across the isthmus with terminals at Panama and Colon. This company did some desultory surveying and sent back to France the astounding announcement that discovery had been made of a trans-isthmian pass only 37 ft. above sea level at Panama. The French government became interested and in 1843 it sent Napoleon Garella to Panama to investigate. His report of course disclosed the fabulous character of the heralded low pass. In addition he made a really careful study, in fact, the first real engineering study of a canal route. His report recommended a lock canal 158 ft. above sea level, to cost some $25,000,000 to $28,000,000, depending on whether the summit was penetrated by tunnel or by open cut. On this report the French company gave up its concession.

In 1847 New Granada granted a concession to the Panama Company, Mateo Klein, agent, to build a railway across Panama. The company failed to secure capital and its concession expired. The Klein concession in modified form was then secured from New Granada by the Panama Railroad Co., incorporated in New York in 1849. This company completed the Panama R. R. in 1855, at a cost of some $8,000,000. This figure is of interest in comparison with the price of nearly $25,000,000 which the French Panama Canal Co. paid in 1881 for the Panama R. R.

In 1850 the American Atlantic & Pacific Ship Canal Co., O. W. Childs, chief engineer, secured a concession for a canal at Nicaragua and completed general surveys in 1852 but, failing to begin construction, lost its concession. In 1869, President Grant in his message to Congress recommended the construction of an isthmian canal by Americans. Congress responded by authorizing the President to appoint an Interoceanic Canal Commission to study the problem and, between 1870 and 1875, surveys were made for canal routes at Tehuantepec, at Nicaragua and at Panama. This first American canal commission consisted of Gen. Andrew A. Humphreys, Prof. Benjamin Pierce and Capt. Daniel Ammen. Later Carlisle P. Patterson succeeded Professor Pierce. The commission's report was made in 1876 and was submitted to Congress in 1879 and it recommended the Nicaragua route.

Trans-isthmian canal activity now turned to France. A canal company was organized in France and in 1876 secured a concession by which, under a later modification, it obtained exclusive right for 99 years to build a canal whose route was to be determined by an international congress. The history of this company constitutes in Isthmian canal history the period of French Canal Construction and is given in a following section. Briefly, the company was sold to Ferdinand de Lesseps in 1879, construction was begun in 1882; the De Lesseps company went into receivers' hands in 1889; the New Panama Canal Co. was organized in 1894 and in 1904 the United States purchased its property.

Meanwhile, in 1884, the United States had again entered the Isthmian canal field by negotiating with Nicaragua a treaty calling for the construction of a canal and involving joint ownership by the two countries. Surveys were made for this canal in 1885 by A. G. Menocal. Nicaragua then withdrew from the treaty. In 1887, however, Mr. Menocal secured from Nicaragua a concession to construct a canal from Greytown to Brito. The Maritime Canal Co. of Nicaragua was organized and was incorporated by Congress in 1889, with a capital of $250,000,000. A construction company with a capital of $12,000,000 was organized and, selling its shares at 50 per cent discount, realized $6,000,000. Work was begun at Nicaragua in 1890 and continued three years, when the $6,000,000 had been spent and the company went into the hands of a receiver.

During the struggles of the Maritime Canal Co. to preserve its existence Congress was repeatedly importuned for aid and finally, in 1895, an amendment to the Sundry Civil Appropriation bill was passed by which a board was created to report upon the feasibility and cost of completing the Panama canal. The board of engineers selected for the work consisted of Col. William Ludlow, Admiral Mordecai T. Endicott and Mr. Alfred Noble. It reported that proper investigation could not be made within the time named and with the funds allotted. Accordingly, in 1897 a new board consisting of Admiral John G. Walker, Col. Peter C. Hains and Prof. Louis M. Haupt was appointed. It submitted its report in 1899. The report advised that a general investigation be made embracing all possible canal routes. In 1899, Congress therefore authorized the *first* Isthmian Canal Commission, a [illegible] members who were named by President [illegible] on March 3, 1899. The work of this [illegible] the period of American [illegible] and [illegible] outlined in a [illegible] section.

Period of French Canal Construction.

Before the Suez Canal was completed in 1869, Frenchmen had begun plans for a canal across the Isthmus of Darien. The leader in these plans was Etienne Türr, a friend and admirer of Ferdinand de Lesseps, the builder of the Suez Canal. He, in 1876, secured through an agent, Anthoine Gogorza, and under date of May 28, a 99-year concession from Colombia authorizing a canal survey and, if this showed a canal without locks or tunnels to be practicable, also authorizing the construction of such a canal. Based on the Gogorza-Türr concession a company was organized and in November, 1876, Lieut. L. N. B. Wyse was sent to Colombia to make surveys. He returned to France in 1877 and again in 1878 went to the Isthmus where, aided by Armand Reclus, he completed his surveys. He also obtained from Colombia a new contract granting exclusive rights to the French company to construct and operate for 99 years a canal whose route was to be determined by an international commission. De Lesseps now first appeared as a factor in the Darien canal project.

On May 15, 1879, largely through the efforts of the builder of the Suez Canal, an international commission of 135 members, 74 of whom were French friends of De Lesseps, was assembled in Paris to decide on a canal route. This commission, 86 members voting, favored by a majority of 70 a sea level canal at Panama from the Gulf of Panama to the Bay of Limon. The cost was estimated at $214,000,000 and the least time of completion as twelve years.

On July 5, 1879, all ownership in the Wyse concession was transferred to De Lesseps, who was to organize as he should see fit a company to carry out the contract. A month later he organized the *Compagnie Universelle du Canal Interoceanique* with a capital of $80,000,000 in $100 shares. Subscriptions for stock were opened in Europe and America, but only some $6,000,000 were subscribed. The project was also so severely attacked that De Lesseps organized an international technical commission of nine members to report on the project and he personally went to Panama to direct the work.

De Lesseps while at Panama with a *"coup de pioche"* (stroke of pick), inaugurated actual work on the *"canal maritime universelle interoceanique,"* while the technical commission undertook its engineering studies. These studies were completed and a report was handed to De Lesseps in about six weeks. This report estimated the cost of the canal at $168,500,000 and the least time of construction as eight years. De Lesseps added a "note" to the report which reduced the estimated cost to $131,600,000. He sailed from Panama to the United States and there began soliciting subscriptions to stock, announcing that he had capitalized his company at $120,000,000. No stock was subscribed in the United States and De Lesseps returned to France. Here in 1880 he opened subscription books for $60,000,000 of stock and it was all taken. A meeting of shareholders was held on Jan. 31, 1881, and on March 3, 1881, the canal company was definitely organized with a capital of $60,000,000.

On June 6, 1881, a detachment of 35 men left France for Panama under the direction of Mr. Armand Reclus, who, with L. N. B. Wyse, had made the original surveys. Construction was not really begun until Jan. 20, 1882. Meanwhile the canal company had found it necessary in order to conduct work to purchase the Panama R. R. The owners of the railway were in position to set their own price and they made the canal company pay $250 for shares having a par value of $100 and worth about par in the market. Controlling ownership of the Panama R. R. cost the canal company over $25,000,000. During the first actual construction year very little real work was done. The general contractors on Dec. 31, 1882, asked to have their contract annulled. De Lesseps consented to this request and appointed Jules Dingler director general of the work. Mr. Dingler arrived at Colon Feb. 27, 1883. His first action was to examine the route and submit a report recommending a canal 29½ ft. deep and 72 ft. bottom width, with a large dam at Gamboa to control the waters of the Chagres. The total excavation was estimated at 157,000,000 cu. yds. This plan was accepted and construction work was divided into seven sections and let to as many contractors, all of whom contracted to complete their work in 1889.

During 1883, 1884 and 1885 canal construction proceeded actively. In 1885, however, the canal company began to experience financial troubles. At a stockholders' meeting on July 29, 1885, De Lesseps admitted that his estimates were too low and that the cost of the canal would reach fully the $214,000,000 estimated by the international congress of 1879. He had already, May 27, 1885, asked the French Government for authority to issue lottery bonds for a sum of $120,000,000. The Government decided not to act on this request until it could secure definite information of conditions at the Isthmus and it sent Mr. Armand Rousseau, an engineer, to examine and report on the canal works. The Government expert went to Panama late in 1885 and returned to France in February, 1886. Simultaneously De Lesseps, with a commission composed of an engineer from Holland and another from Germany and of representatives from the Chambers of Commerce of leading French cities, and with others, sailed for Panama. The visit lasted two weeks, and on the return of the party to France favorable report was made by the delegates from the Chambers of Commerce. The engineers of the De Lesseps' commission made no statement. The French Government expert, Mr. Rousseau, in the meantime had reported that the completion of the proposed canal was impossible unless there was a change to the lock plan. This report was supported by Leon Boyer then director general of the canal work and by Mr. L. Jacquet, one of the canal company's engineers.

The next step taken by De Lesseps was to withdraw his request for authority to issue lottery bonds and to obtain permission from the canal company stockholders to issue a new series of company bonds. Bonds were issued and sold in 1886 amounting to $70,000,000. This sum was too small to complete the sea level canal as planned and in July, 1887, De Lesseps announced that a new plan providing for a lock canal was under consideration. In October, 1887, a commission appointed by De Lesseps reported that such a plan was possible and that the canal could be completed in 1891.

Following this report and change of plan the French Government granted permission to the canal company to issue lottery bonds. An issue of 2,000,000 bonds was made but after two offers, only 1,000,000 were taken. On Dec. 14, 1888, De Lesseps asked the courts to appoint temporary managers for the company. The temporary managers after unsuccessful effort to reorganize, reported to the stockholders advising the appointment of a receiver. This appointment was made Feb. 4, 1889. The investigation of the company's affairs showed that nearly $263,000,000 had been collected and expended by the company and that a total of 66,743,551 cu. yds. of material had been excavated. Various extravagant expenditures were revealed and general and frequent bribery and misappropriation of funds were proved. Ferdinand de Lesseps and his son Charles, with some others of the canal company's officials, were sentenced to fine and five years' imprisonment. The elder De Lesseps escaped imprisonment by death and the sentence against Charles de Lesseps was annulled by the Court of Appeals.

The receiver of the canal company stopped canal work in May, 1889, and sent a commission to Panama to investigate and report on the work. A report was received in May, 1890, which stated that a lock canal could be constructed at an estimated cost of $180,000,000 and that the value of the plant and of the work done was some $90,000,000. From the date of this report until the fall of 1894 work was confined to securing from Colombia extensions of contract, to the organization of a new company and to capitalizing the company. The new canal company was finally organized Oct. 20, 1894, with a cash capital of $12,000,000. Construction was resumed in 1894. In 1896 an international commission was appointed by the company to make studies for canal plans and it made a complete and careful investigation lasting three years; its report recommended a canal, with five locks on each slope, dams at Bohio and Alhajuela, a depth of 29½ ft., and a bottom width of 98 ft., to cost $102,000,000. These plans were adopted by the company which was working on them at the time of the transfer of the work to the United States on May 4, 1904.

The new French Panama Canal Co. excavated some 11,400,000 cu. yds. mostly in Culebra cut. In summary, the French Panama Canal companies during the period March 3, 1882, to May 4, 1904, or from the time De Lesseps' original company was organized till the New (French) Panama Canal Co. turned over its property to the United States, excavated some 78,146,960 cu. yds. and spent some $270,000,000. Their hospital records showed some 6,283 deaths and the unrecorded death list was about twice the number of recorded dead. In round figures it is estimated that 20,000 lives were lost during the French work at Panama.

Period of American Canal Planning.

Active planning of the present Panama Canal began with the studies of the Isthmian Canal Commission created by Congress in June, 1899. This commission consisted of Rear-Admiral J. G. Walker, Hon. Samuel Pasco, Mr. Geo. S. Morison, Mr. Alfred Noble, Lieut. Col. O. H. Ernst, Col. Peter C. Hains, Prof. L. M. Haupt, Prof. Wm. H. Burr, Prof. Emory R. Johnson and Lieut. Com. Sidney A. Staunton, Secretary. The commission completed its report in November, 1901. During its 2½ years' work it made studies of 19 possible trans-isthmian canal routes and of these, four routes were investigated in detail. The report was a voluminous and comprehensive document and was replete with valuable information. Lack of space prevents here mention of any but the main features.

Two routes, the Panama route and the Nicaragua route, recommended themselves to the commission. Choice of the Panama route would necessitate the purchase of the French Panama Canal Company's interests. An evaluation by the commission of the French property was as follows:

Useful canal excavation	$21,040,386
Chagres diversion	178,186
Gatun diversion	1,396,456
Railroad diversion	300,000
Contingencies	4,579,005
Panama R. R. stock	6,830,009
Maps, drawings and records	2,000,000
Total	$36,324,033
Add 10 per cent to cover omissions	3,675,967
Grand total	$40,000,000

Whether this estimated value was actually received when the property was afterwards purchased by the United States has been so often questioned that a statement of final appraisal made in 1911 is given in a separate section.

The French canal company set a price on its property of $109,141,000. The commission reported that a lock canal was obligatory at Nicaragua but that either a sea level or a lock canal was possible at Panama. The commission recommended a lock canal at Panama should that route be chosen. It estimated the cost of a lock canal at Panama at $144,233,358. Add the price, $109,141,000, asked by the French company and the cost would be $253,374,858. The commission estimated the cost of the Nicaragua canal at $189,684,062. It, therefore, recommended the Nicaragua route. Immediately upon this report, the French canal company cabled to the United States an offer to sell its property for the commission's estimate of $40,000,000. The commission thereupon issued a supplementary report, dated June 18, 1902, to the effect that "after considering the changed conditions that now exist, the commission is of the opinion that 'the most practicable and feasible route' for an Isthmian canal, to be 'under the general management and ownership of the United States is that known as the Panama route.'"

On June 28, 1902, the so-called Spooner bill passed by Congress, became effective and it authorized: (1) The purchase of the rights and property, including the Panama R. R., of the New (French) Panama Canal Co.; (2) acquiring from the Republic of Colombia perpetual control of a strip of land for the canal with all necessary water rights, right to exercise sanitary control, etc.; (3) the actual building of the canal; (4) the making of arrangements for building a canal on the Nicaragua route should negotiations with the French canal company or with the Colombian Government fail, and (5) the creation of an Isthmian Canal Commission of seven members, four of whom were to be skilled engineers, and of the four engineers, one was to be an officer of the Navy and one an officer of the Army. The Spooner act also appropriated $10,000,000 to be immediately available, $40,000,000 for the purchase of the French canal property, a further sum of $135,000,000 in case of selection of the Panama route or of $185,000,000 in case the Nicaragua route was selected.

The details of the treaty agreed to by Colombia June 22, 1903, its repudiation by Colombia in August, 1903; the assertion of the independence of Panama Nov. 4, 1903; the canal treaty negotiated with Panama Feb. 23, 1904, are political rather than engineering matters and are merely scheduled. On May 4, 1904, purchase by the United States of the New Panama Canal Co. (French) property was completed. The United States had previously paid to Panama for the Canal Zone, $10,000,000. The construction period of the American Panama canal was inaugurated by the appointment of a new Isthmian Canal Commission conforming to the requirements of the Spooner act.

Work of the Second Isthmian Canal Commission.

The first commission designated as Isthmian Canal Commission was the commission of nine members which decided upon the canal route. The Spooner act authorizing canal construction required a commission of seven members and gave the appointment of these members to the President with very wide privileges in defining the duties and relative authority of the several commissioners. President Roosevelt appointed the second Isthmian Canal Commission Feb. 29, 1904, giving all commissioners the same authority and the same compensation of $12,000 per year. The commissioners appointed were: Admiral John G. Walker, General Geo. W. Davis, William Barclay Parsons, William H. Burr, Benj. M. Harrod, Carl E. Grunsky, and Frank J. Hecker. These appointments were confirmed by the Senate on March 3, 1904. In May, 1904, the commission appointed Mr. John F. Wallace as its chief engineer, at a salary of $25,000 a year.

On May 9, 1904, President Roosevelt placed the work of the commission under the direction of the Secretary of War. On the same date he gave the commission power to legislate for the Canal Zone and appointed General Davis, one of the members, Governor of the Zone. The commission sailed for Panama March 29, 1904, and remained until April 20, 1904. There went with the commission as experts on sanitation Col. William C. Gorgas and Maj. Louis La Garde, Medical Corps U. S. A.; Dr. John W. Ross, Medical Director, U. S. N., and Capt. C. E. Gillette, Corps of Engineers, U. S. A. These experts made a study of health conditions at the Isthmus and formulated a plan for the sanitation of the Zone and of the cities of Colon and Panama and this plan was adopted by the commission on June 2, 1904. Colonel Gorgas was appointed chief sanitary officer and instructed to proceed with the work of sanitation as planned. Colonel Gorgas reported at Panama and on June 30, 1904, Governor Davis announced the organization of the Sanitary Department with Colonel Gorgas at its head. On Sept. 2, 1904, the Department of Public Health was created with Colonel Gorgas, chairman, and Dr. John W. Ross, Dr. H. R. Carter and Joseph Le Prince, members. This board was under authority of and reported to the Governor of the Canal Zone. The work of this board was of such exceptional character and so important a factor in the success of the canal work that it is given separate consideration elsewhere.

While the task of sanitation was being planned and organized the work of organizing civil government of the Canal Zone was proceeded with by Governor Davis and the Commission. During 1904 a Zone Government was established consisting, besides the Governor, of an Executive Secretary, a Treasurer and an Auditor; a judicial department consisting of a supreme court, three circuit courts and five municipal courts; departments of public health, revenues, police and prisons; a postal service; a bureau of education and a school system. A penal code was enacted, a code of criminal procedure, and laws suppressing lotteries and prohibiting gambling.

Progress with actual construction was attempted simultaneously with the prosecution of sanitary work and with the organization of a Canal Zone Government. Chief Engineer John F. Wallace assumed office at Panama July 1, 1904. There were then at work in Culebra Cut about 700 of the French Panama Canal Company's old employes using old French plant in poor condition. There were scattered along the route and buried in the jungle numerous kinds of plant and an immense amount of railway material all out of date and much of it useless. The machine shops were in bad repair, inadequately equipped with out-of-date machinery. Some 2,000 buildings capable of repair and use, but useless in their present condition, existed. The Panama R. R. was in poor condition and provided with antequated rolling stock. There was no adequate food supply.

The commission and the chief engineer undertook to begin actual excavation under these conditions. During the year there was gathered a working force of about 8,000 men; some 350 buildings and the old machine shops were repaired and enlarged; the construction of water works for Panama and Colon was begun; wharves and terminals were repaired and enlarged, and some 742,000 cu. yds. of excavation were accomplished. Plans were also worked out for an adequate food supply.

These material accomplishments of the commission in one year were important but they were at the expense of much friction. The commission of seven executives of equal power did not act as a unit. On Nov. 16, 1904, Mr. Frank J. Hecker resigned from the commission because of "its ineffectiveness." Mr. William Barclay Parsons announced to President Roosevelt his intention of resigning because the commission was so organized as to prevent successful executive action. On Jan. 12, 1905, Secretary of War Wm. H. Taft, in a letter to President Roosevelt, designated the commission as not well adapted to perform the task before it. General Davis, Governor Canal Zone, stated by letter to Secretary Taft that "the present body is far and away too cumbersome and ineffective." Chief Engineer Wallace and Dr. Gorgas registered repeated complaints of the incompetency of employes sent to Panama, and of delay and failure in granting requisitions for necessary plant, materials and supplies.

On March 4, 1905, President Roosevelt asked the members of the commission for their resignations, and on April 1, 1905, he appointed a new commission.

Work of the Third Isthmian Canal Commission.

The members of the Isthmian Canal Commission appointed April 1, 1905, were: Theodore P. Shonts, chairman; Charles E. Magoon, Governor Canal Zone; John F. Wallace, Chief Engineer, and Admiral Mordecai T. Endicott, General Peter C. Hains, Col. Oswald H. Ernst and Benjamin M. Harrod, members. On April 5, 1905, President Roosevelt issued an executive order defining the duties and powers of the members of the commission as follows: Chairman, general direction and specific direction of making contracts and purchasing supplies; Governor Canal Zone, administration and enforcement of law in the Canal Zone and general control of all sanitation; Chief Engineer, full charge of all construction work on the Isthmus. The four remaining members of the commission were constituted an advisory engineering committee. On June 28, 1905, Chief Engineer Wallace resigned his office and on July 1, 1905, Mr. John F. Stevens was appointed Chief Engineer.

Mr. Stevens assumed charge at Panama on July 26, 1906, and after a study of conditions announced that effort should be concentrated on improving the Panama R. R. and its ocean terminals, on planning a transportation system for handling excavation, on the construction of shops, workmen's quarters, on sanitary works and on collecting plant, materials and supplies for construction, and that actual excavation should be confined to preparing Culebra cut for the installation of excavating machinery and spoil transportation tracks. Following this advice the commission in July, 1905, directed that all excavation be discontinued except such work in Culebra cut as the Chief Engineer should deem necessary to prepare for plant and tracks.

The preparatory work accomplished can best be indicated by the sums expended for different purposes. These were: For Zone government and sanitation, about $5,000,000; for construction quarters and other buildings, docks, railway improvements, water supply and sewerage and canal construction, about $7,000,000; for permanent plant about $12,000,000; for materials and supplies, about $4,500,000, and for city improvements in Colon and Panama about $1,500,000. The results are summarized by Mr. Joseph Bucklin Bishop, Secretary Isthmian Canal Commission, in "The Panama Gateway" as follows:

> At the end of that time the Isthmus had been converted into as healthful a place of abode as could be found anywhere in the tropics and more so than many places outside the tropics; yellow fever had been banished for all time; the harmful activities of malarial mosquitoes had been greatly restricted; adequate quarters, comfortably furnished, had been provided for all employes; a system for the abundant supply of pure food at moderate prices, brought from the United States, had been established and was working well, an ample supply of pure water had been secured for all cities, towns, villages and camps, the Panama Railroad had been double-tracked for the greater part of its length and furnished with new and heavier rails and a modern personnel and equipment; terminal railway yards and two great intermediate forwarding and receiving yards for dirt-trains had been constructed; new wharves at the terminals on the Atlantic and Pacific, with modern unloading machinery, had been built and the harbors about them dredged, a thorough system for receiving, handling, and distributing the vast amount of supplies which came on every steamer had been created and put in operation; great [illegible] shops with modern appliances, had [illegible] red from the old ones, and in these [illegible] locomotives, cars, steam-shovels and [illegible] of the canal plant which arrived [illegible] parts were set up or put [illegible]
>
> [illegible] arrived and was installed in [illegible] me of excavation were ex[illegible] 1, 1906, the working force [illegible] en and was being in[illegible]. As the type of canal [illegible] de[illegible] excavat[illegible]

a large scale was not possible, so on June 24, 1905, President Roosevelt appointed an International Board of Consulting Engineers to consider the various canal plans proposed and to decide upon the type of canal to be built. This board was composed as follows: Geo. W. Davis, Major General U. S. A., and member First Isthmian Canal Commission; Alfred Noble, Chief Engineer, East River Tunnels, Pennsylvania R. R.; William Barclay Parsons, Chief Engineer, Rapid Transit Commission, New York, and member First Isthmian Canal Commission; Wm. H. Burr, Professor of Civil Engineering, Columbia University, and member First Isthmian Canal Commission; Henry L. Abbot, Brigadier General, U. S. A.; Frederick P. Stearns, Chief Engineer, Metropolitan Water and Sewerage Board, Massachusetts; Joseph Ripley, Chief Engineer, Sault Ste. Marie Canal; Isham Randolph, Chief Engineer, Chicago Drainage Canal; William Henry Hunter, Chief Engineer Manchester Ship Canal; Adolph Guérard, Inspector Bridges and Roads, France; Eugen Tincauzer, Chief Engineer Kiel Canal, Germany; J. W. Welcker, Chief Engineer Waterstaat, Netherlands; Edouard Quellennec, Consulting Engineer, Suez Canal.

The Board of Consulting Engineers assembled in Washington Sept. 1, 1905; it went to Panama in October and made careful investigation of the canal route and it also visited important American canal works. On Nov. 18, 1905, the board by formal vote of eight members against five members passed a resolution favoring a sea level canal. Both majority and minority prepared reports and the final and formal report was handed to President Roosevelt on Jan. 10, 1906. The document presented a majority report, signed by Messrs. Hunter, Welcker, Quellennec, Guérard, Tincauzer, Burr, Parsons and Davis, recommending a sea level canal and a minority report signed by Messrs. Ripley, Noble, Randolph, Stearns and Abbot recommending a lock canal. A comparison of the two plans, based on the report is made in another section. The two reports were referred to the Isthmian Canal Commission which handed them back Feb. 5, 1906, with a report approving the lock canal plan of the minority report. Chief Engineer John F. Stevens concurred in this decision and on Feb. 19, 1906, Secretary of War Taft returned the several reports to President Roosevelt with a report approving the report of the Isthmian Canal Commission. President Roosevelt sent all documents to Congress where the question of type of canal was discussed with great difference of opinion until June 21, 1906, when the Senate by vote of 36 to 31 approved the lock type. On June 27 the House of Representatives concurred and on June 29, President Roosevelt signed the act for a lock canal at Panama.

With the type of canal decided and with improved conditions for work on the Isthmus, excavation could be pushed, and during the last six months of 1906 about 1,500,000 cu. yds. of earth and rock were removed from Culebra Cut. This brought the total excavation of 1904-5-6 to about 7,000,000 cu. yds. with everything in readiness for active excavation on a large scale in 1907. A change of commission was necessitated, however, before work could go ahead.

In October, 1906, Mr. Chas. E. Magoon was transferred from the Canal Zone to Cuba as Provisional Governor. Colonel Ernst had already been transferred to the Mississippi River Commission. Their places on the Canal Commission had not been filled. Early in 1907 Chairman Shonts offered his resignation to take effect March 4. Chief Engineer Stevens a little later asked to be relieved and his request was granted with the stipulation that he should remain until his successor could become familiar with the work. On Feb. 26, 1907, President Roosevelt directed the chairman to assign Col. Geo. W. Goethals, Corps of Engineers, U. S. A., to the office of chief engineer. Chairman Shonts' resignation went into effect March 4. Chief Engineer Stevens was appointed to Mr. Shonts' place. On March 16, 1907, Messrs. Hains, Harrod and Endicott resigned. Mr. Stevens' retirement was set for April 1, and on this date a new commission appointed by President Roosevelt on March 7, assumed charge of the work.

Proposals for Building Canal by Contract.

The second Isthmian Canal Commission began construction by day labor and the third commission continued this method largely because it was not ready to undertake any other method until the Isthmus had been prepared for habitation and labor. A sentiment had been manifest from the first in the United States in favor of canal construction by contract. During the year of the third commission's work it had increased in force. Within the commission, Chairman Shonts and Chief Engineer Stevens were in favor of construction by contract and on Aug. 29, 1906, Chairman Shonts recommended to the Secretary of War that bids be asked on a special form of contract which was submitted by him. This contract was a voluminous document and only a few of its salient features can be stated.

It called for the construction of a canal of the dimensions recommended by the Minority Report of the International Board of Consulting Engineers. Any association of contractors might submit proposals providing it had an available capital of $5,000,000 in excess of liabilities. Bidders were to submit with their bids a certified check of $200,000, be prepared if awarded the contract to give bonds for $3,000,000 and agree to enter into contract within 15 days. The commission was to furnish substantially all plant except minor tools, was to furnish all construction materials and was to supply fuel or electricity for power of all kinds. It was also to supply free of charge existing construction tracks, quarters, warehouses, hospitals, office buildings and other facilities, and it was to retain control of all engineering works, municipal engineering, police, hospital service, commissary service, and the like.

The contractor was to receive monthly payments covering the cost of the actual construction work of the preceding month and also two-thirds of the agreed percentage of the actual cost of work done each year. Upon the completion of the work the balance on the percentage allowance was to be paid with certain reductions or additions. In case the actual cost exceeded the estimated cost, a deduction of 5 per cent from the percentage otherwise payable for each $5,000,000 or half fractional part thereof, of actual cost in excess of estimated cost; in case the actual cost was less than the estimated cost, an addition to the agreed percentage, of one-third of the difference between the actual cost and the estimated cost, was to be paid. If the work was not completed within the stipulated time limit, the contractor was to forfeit as liquidated damages $100,000 for each month of delay, but for every month gained by the contractor on the time limit he was to be allowed $100,000 per month.

The cost and time estimates were to be made, after the award of the contract, by a committee of five engineers, one being the chief engineer of the commission, two being appointed by the commission and two being appointed by the contractor.

Bids on this contract were received on Jan. 13, 1906, and were as follows:

Firm.	Percentage.
Geo. Peirce & Co.	7.19
Wm. J. Oliver and Anson M. Bangs	6.75
McArthur-Gillespie Co.	12.50
North American Dredging Co.	28

On Feb. 26, 1907, the Isthmian Canal Commission, under instructions from President Roosevelt, rejected all bids. The reasons for rejection were stated by the President as follows:

> There were two bids worthy of consideration. The bid of the McArthur syndicate at 12.5 was the only one which came within the requirements. The Oliver and Bangs bid at 6.75 was rejected as not satisfying the specifications of the invitation.

Mr. McArthur and his associates not objecting, and preferring this course to an invitation for new bids, Mr. Oliver was allowed to perfect his bid with new associates and new financial responsibility; but this permission did not in any way change the situation from what it would have been had Mr. Oliver's bid in its present form been presented on Jan. 13, the day fixed for receiving the bids. Under the invitation for bids the reserved power exists in the commission to reject all bids, and the first question is whether it shall do so.

The purpose of the contract was to secure in the building of the canal the services of the best, most experienced and most skilled contractors in the country at the least risk to them, and at the least expense to the government. An investigation into the two bids above mentioned shows that this purpose of the government has failed.

In each bid the contractors of experience, whose personal services in the work are what the commission has sought, have made arrangements to divide the profits under the percentage bid with bankers or others to whom the contractors have had to look for the needed capital, so that the contractors who are actually to do the work have arranged to accept a comparatively small proportion of the profits accruing under the contract.

In other words, the government by this arrangement is made to pay a high percentage for the use of capital which it might itself have furnished at a much lower rate, while the percentage which the contractors are to receive for the real benefit they are to confer on the government is reduced to very meager and perhaps inadequate compensation. No contract can ultimately operate to the benefit of the government in which the contractor's energy, skill and personal supervision of the work are not adequately paid for.

The defect in the bids which I have described, and which our investigation has made apparent may be due to a defect in the invitation for bids, which perhaps stipulated for too heavy a bond and the investments of too large capital by the contractors, or it may be because the bidders have taken an entirely different view of the money risk involved from that taken by the government.

Mr. Stevens, the chief engineer, who proposed the form of contract which, with modifications, was adopted, advised against accepting either bid, because acceptance of either would not, in his judgment, accomplish the purpose he sought. The considerations stated are by themselves sufficient to require a rejection of all bids and a change in the proposed form of contract.

One of the chief reasons for adopting the contract as proposed was that in its main features it was formulated by Mr. Stevens, who was expected to supervise the work as chief engineer. He had had experience with contracts of this character, and he had had eighteen months' actual experience with the work on the Isthmus. Less than two days ago I received a letter from Mr. Stevens, in which he asked to be entirely relieved from work on the canal as soon as he could be replaced by a competent person and that person could become familiar with the work.

I have accepted his resignation. The withdrawal of Mr. Stevens takes away the special reason mentioned for proceeding under the present form of contract.

Following Mr. Stevens' resignation a new commission was appointed as is noted in the succeeding section and construction by the day labor plan was finally adopted.

Work of the Fourth Isthmian Canal Commission.

The fourth Isthmian Canal Commission was appointed on March 7, 1907, and it assumed its duties April 1, 1907. The members of the commission were: Lieut. Col. Geo. W. Goethals, Chairman and Chief Engineer, and members: Major D. D. Gaillard, Major William L. Sibert, Civil Engineer H. H. Rousseau, U. S. N., Col. W. C. Gorgas, J. C. S. Blackburn, and Jackson Smith. Joseph Bucklin Bishop was made secretary to the commission. The chairman's salary was made $15,000 per year and the other members each received $14,000 per year. On April 10, 1910, Mr. Blackburn was succeeded by Maurice H. Thatcher, and on Aug. 8, 1913, Mr. Thatcher was succeeded by Richard L. Metcalf. Mr. Jackson Smith was succeeded on July 16, 1908, by Lieut. Col. H. F. Hodges. The commission was distinctly a commission of army engineers, and indeed it was the avowed intention of President Roosevelt that it should be.

The work of this commission has been the construction of the Panama Canal and this task obviously cannot be scheduled as has been the smaller accomplishments of the preceding commissions. There will be given here, therefore, merely an outline of the changes of organization adopted by the commission for handling its task.

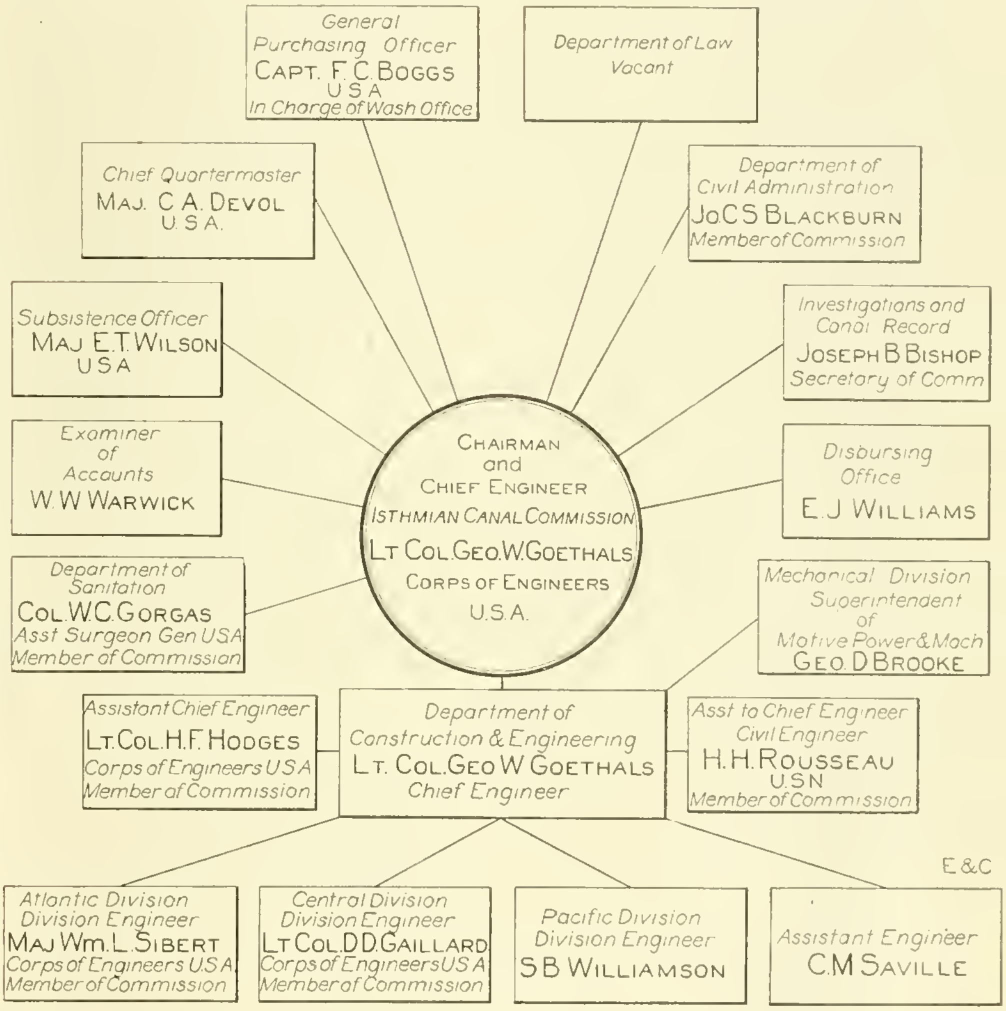


Fig. 1. Diagram Showing Organization for Canal Construction.

On July 14, 1908, Colonel F. H. Hodges was appointed to succeed Mr. Jackson Smith, and assumed his duties on that date. At the time the fourth commission assumed office the Division of Construction and Engineering consisted of three departments and of twelve divisions; each division had its independent administrative organization and reported through the head of the department to which it belonged. To concentrate authority a new organization for work was inaugurated July 1, 1908, and was completed during the following year. Three construction divisions were established as follows: Atlantic Division extending from deep water in Limon Bay to and including the Gatun locks and dam; Central Division extending from Gatun Dam to the Pedro Miguel locks; Pacific Division extending from the Pedro Miguel locks inclusive to deep water in Panama Gulf. All work on building construction and maintenance was placed in charge of the Quartermaster's Department. All sanitary engineering works were placed under the direction of the division engineers. A mechanical division reporting directly to the chief engineer, and having charge of motive power and machinery was organized. The quartermaster's department also assumed the work of recruitment of labor, the handling of materials and supplies and sanitary policing. A Department of Subsistence was created to manage hotels, kitchens, and commissary. The general organization on June 30, 1909, and the names of the directing officials are given by the chart, Fig. 1. On Dec. 4, 1909, Commissioner Blackburn resigned, and on April 12, 1910, his position was filled by Mr. M. H. Thatcher. The vacant position, shown in Fig. 1 as head of the department of law, was filled by the appointment of Mr. Frank Feuille on April 16, 1910. On Jan. 19, 1911, a Canal Zone Land Office was created to handle records, maps and all documents and affairs having to do with canal lands and lands of the Panama R. R. On May 12, 1911, Mr. H. A. A. Smith was appointed examiner of accounts to succeed Mr. W. W. Warwick, transferred to other duties. On Jan. 1, 1912, a department of fortifications was created and placed in charge of Lieut. Geo. R. Goethals. With the changes noted the organization for directing work remained as indicated by the chart, Fig. 1, until Jan. 1, 1913, when Mr. S. B. Williamson resigned, and the work of the division was divided and transferred to various divisions of the chief engineer's office.

The preceding outline touches only a few general features. A record in detail of the changes and modifications introduced from time to time would entail excessive space and would serve no important purpose. In a force as large as the Panama Canal construction required, there are necessarily many engineers whose work has been of the utmost value, but who cannot well be singled out for individual mention.

Evolution of Plans for a Canal at Panama.

Physical Characteristics of the Panama Canal Route.

At Panama the northwest-southeast axial line of the Isthmus of Darien is interrupted by a loop to the north. This loop is roughly semicircular and the two coast lines are nearly parallel. The concave or Pacific side of this loop forms the Gulf of Panama. On the convex or Atlantic side is the Bay of Limon on which the city of Colon is located. This general geography of the Isthmus at Panama is indicated by the small scale map of Fig. 2. From the large scale map, Fig. 2, it will be seen that Colon on the Atlantic side is some 20 miles farther west than Panama on the Pacific side and that a line connecting the two is northwest-southeast. Such a line it will also be noted crosses the axis of the Isthmus nearly at right angles; also such a line crosses the Isthmus at about its least width and where the elevation of the continental divide is the lowest.

The two slopes of the continental divide at Panama are drained by the Chagres River and the Rio Grande and the axes of these rivers from the points where they debouch from the highlands and turn toward the Atlantic and the Pacific, respectively, are approximately on the northwest-southeast line drawn across the Isthmus as described above. These rivers were naturally a prominent feature in any plan for a trans-isthmian canal at Panama, and they will be described somewhat carefully.

The Chagres River drains the Atlantic slope; it is the largest river of Panama and has a drainage basin of some 1,350 square miles. Its source is in the San Blas Mountains to the northeast of the Canal Zone and its course is generally parallel to the Carribean Coast as far as Bas Obispo, which is at the mouth of the Obispo River. Here the Chagres turns northwest almost at right angles and takes a winding but consistently northwestern course to the Carribean Sea. From its source to Bohio the Chagres traverses a comparatively narrow valley with rather steep sides; at Bohio its waters have descended to an elevation but little above sea level and its course is winding and passes through lowlands and swamps.

The Rio Grande on the Pacific slope is roughly similar in character to the Chagres but is a much smaller stream and is a pronouncedly tidal stream. Its lower reaches, nearly half the length of the river, are a tidal estuary filled and emptied twice a day by the tides.

Proceeding across the Isthmus from the Atlantic to the Pacific the essential geography and topography are as follows, reference being had to the topographic map, Fig. 3. The Carribean entrance to the canal is the Bay of Limon, a curiously rectangular bay about three miles wide and five miles long. This bay has a depth of water varying from 5 ft. to 36 ft., and a tidal oscillation of 2 ft. Colon occupies a small peninsula on the east shore of the Bay of Limon and near its entrance. The land along the east shore of the bay is low and swampy. Bordering the west shore is a line of hills. These highlands continue around the south end of the bay but at a much less height and they are nearly cut through by the valley of the small Mindi River. This valley debouches into the swamp lands along the eastern shore of the bay. The Chagres coming from the southeast toward Limon Bay is turned to the northwest by the low ridge which terminates the Mindi valley. The level of the Chagres south of the separating ridge is, however, not much higher than that of the Mindi River and of the swamps along the east shore of Limon Bay and this level continues to Bohio some 16 miles inland.

The main tributaries of the Chagres are the Gatun or Gatuncillo River and the Trinidad River. The Gatun enters the Chagres from the northeast. The Trinidad enters the Chagres from the southwest and farther inland. At Bohio the broad swampy valley of the Chagres rapidly narrows and its grade increases. At Bas Obispo, 13 miles inland from Bohio and 29 miles from Colon, the Chagres turns northeast into the mountains. Here the level of the Chagres is 45 ft. above sea level and here also begins the ridge of the continental divide. However, this ridge is cut into by the valley of the Obispo River which extends from Bas Obispo directly toward the Pacific.

The continental divide extends from Bas Obispo to Pedro Miguel, a distance of nine miles. There is a low summit at Empire and a high summit at Culebra, where the ground elevation on the center line of the canal is 312 ft. above sea level. Culebra is about six miles from Bas Obispo and about 35 miles from Colon. From Culebra the land level begins to fall toward the Pacific and the descent is much more rapid than that toward the Atlantic; at Pedro Miguel, three miles from Culebra, the level is lower than at Bas Obispo nine miles back. From Bas Obispo to Pedro Miguel is then the Culebra cut, and from Las Cascadas to Pedro Miguel, six miles, is the deepest part of this cut.

From Pedro Miguel to Miraflores, 1½ miles, the descent is more gradual and the valley of the Rio Grande is followed. At Miraflores the Rio Grande valley is but little above sea level and is bordered by highlands on each side nearly to the Gulf of Panama. The distance to the gulf shore from Miraflores is about 5½ miles and the distance from the Bay of Limon is 35 miles.

The Bay of Panama has a depth of water of from 7 to 32 ft., and its bottom slopes to deep water in the Pacific. Its tidal range is about 20 ft. Its only other geographical feature of importance in connection with the canal is the group of small islands lying offshore.

Modification of Natural Features By the French Canal Works.

The original French plans were for a sea level canal. Later when the plan was changed to a lock canal the first set of locks from the Atlantic side was located at Bohio and to this point from Colon the canal was at sea level. The route of the canal as indicated by Fig. 2 was through the swamps along the east shore of the Bay of Limon and in a nearly direct line to Gatun. From Gatun to Bas Obispo the line followed the Chagres, whose loops it crossed repeatedly by its more direct channel. From Bas Obispo to the Pacific the route was essentially that of the present canal. At the time of American occupation of the Canal Zone, the French canal had been excavated from Colon to Gatun and was navigable. The depth of the excavated channel did not exceed 20 ft. It cut the Chagres at Gatun and received about one-third of the flow of that stream. The Gatun River was cut off from the Chagres by a diversion about eight miles long to the Bay of Limon. The large bend in the Chagres near Gatun was also eliminated by a cut-off known as the Chagres Diversion. There were also a number of other such diversions. From Gatun to Bohio the French canal had been excavated to a portion of its depth but nowhere was the depth below sea level. Through the divide (Culebra Cut) the excavation had been carried at its deepest point to 165 ft. below the ground surface. On the Pacific slope a partial channel had been dredged down the Rio Grande and various diversions of inflowing streams had been effected. All along the French canal cut the channel had been silted up in places and the excavated spoil occupied the line of the American canal in many places. Broadly speaking all the excavation performed for the French canal that was particularly useful was: (1) the channel from Colon to Gatun which was used for shipping in construction material for the Gatun locks and spillway, and (2) the excavation completed at Culebra Cut.

Valuation of the French Canal Property.

The first Isthmian Canal Commission appraised the property of the New (French) Panama Canal Co. at $40,000,000, itemized as previously stated, and $40,000,000 was the purchase price paid May 4, 1904, by the United States. To record the value of this purchase definitely the Isthmian Canal Commission on May 3, 1911, appointed a special committee to make exact appraisal and the main parts of this appraisal were as follows:

For the project, which was authorized in its general features in June, 1906, the volume of useful French excavation is much less than that estimated for the project of the Commission of 1899-1901, chiefly because a very large portion of the French canal between Gatun and Bas Obispo is below the + 40 ft. grade of the present project. The committee of December, 1909, appointed to determine the volume of French excavation, which was useful to the present project. reported the amount as 29,908,000 cu. yds., classified as follows:

	Cu. yds.
Dry	23,138,000
Wet	6,770,000
Total	29,908,000

It was decided by the committee to value this excavation at the average price of excavation under the present Commission previous to June 30, 1909, at which time a total of about 40,000,000 cu. yds. of dry excavation had been removed. This average price was $1.03 for dry excavation, and 23 cts. for wet. On this basis the value of useful French excavation to the present project is:

Dry, 23,138,000 cu. yds. at $1.03	$23,832,140
Wet, 6,770,000 cu. yds. at $0.23	1,557,100
Total	$25,389,240

Included in the purchase from the new Panama Canal Co. were 68,888 shares of Panama R. R. stock, par value $100 each, leaving 1,112 shares in the hands of private parties. These 1,112 shares were purchased at a cost of $157,118.24, or an average price of $140. In arriving at the value to the Commission of the 68,888 shares, the average price paid for the outstanding shares was used. This gives a valuation of $9,644,300. When it is considered that the railroad is a very valuable asset to the Commission, and, that through ownership of the property, the Commission has secured the transportation of its freight and passengers at cost, besides using the Panama R. R. Co. in many other ways, the valuation was thought reasonable.

After careful consideration of maps, drainage and records based on experience since the Americans began work, it was the unanimous judgment of the committee that the estimate of $2,000,000 was a conservative one, and should stand approved in the present valuation.

A partial inventory of the material and equipment received from the French was made by the Commission in June, 1906, and this has been supplemented with the following results:

Floating equipment	$ 651,000
Shop machinery and tools	111,076
Rolling stock	297,900
Scrap	294,071
Miscellaneous material	751,396
Air compressors	6,620
Total value	$2,112,063

An estimate of the value of the buildings received from the French company, based on the past value of the buildings at the time of the transfer, plus the value of repairs, minus 10 per cent to cover depreciation, was made by the architect of the Commission in 1908. There were 2,148 buildings in the Canal Zone turned over by the French, of which, on Aug. 1, 1908, 1,536 had been repaired and were in use. Their estimated value was as follows:

Quarters, gold	$ 625,483.63
Quarters, silver	443,800.30
Hotels	7,455.23
Hospitals	482,763.31
Jails	13,503.34
Schoolhouses	12,702.78
Shops	11[illegible],200.8[illegible]
Storehouses	91,099.70
Structures	[illegible]7,218.[illegible]
Miscellaneous buildings of the Department of Sanitation and Government	11,976.46
Total	$1,879,203.84

Not included in this list was the Administration building in Panama, and the residence at present used as the American Legation in Pan- then equivalent to $428,096.58 gold. This land ama. The value of the former, in the judgment was included in the property transferred to the of the Committee, was $125,000; and of the [illegible] making the [illegible] buildings received from the French $2,[illegible].

The French canal company purchased during its existence a total of 13,520 hectares of land, for which it paid $525,120.73 Colombian money. United States, but no estimate of its value was made by the Commission of 1899-1901. The price paid for those portions of these lands lying along the line of the canal does not vary materially from the prices paid by the United States for lands which it has purchased in similar localities since construction began, but others of the French lands, notably those lying near the Pacific entrance of the canal, were purchased at much lower figures than they could be purchased today. Taking as a basis for valuation the prices which the United States has paid for lands, both through private agreements and under awards made by joint commissions,

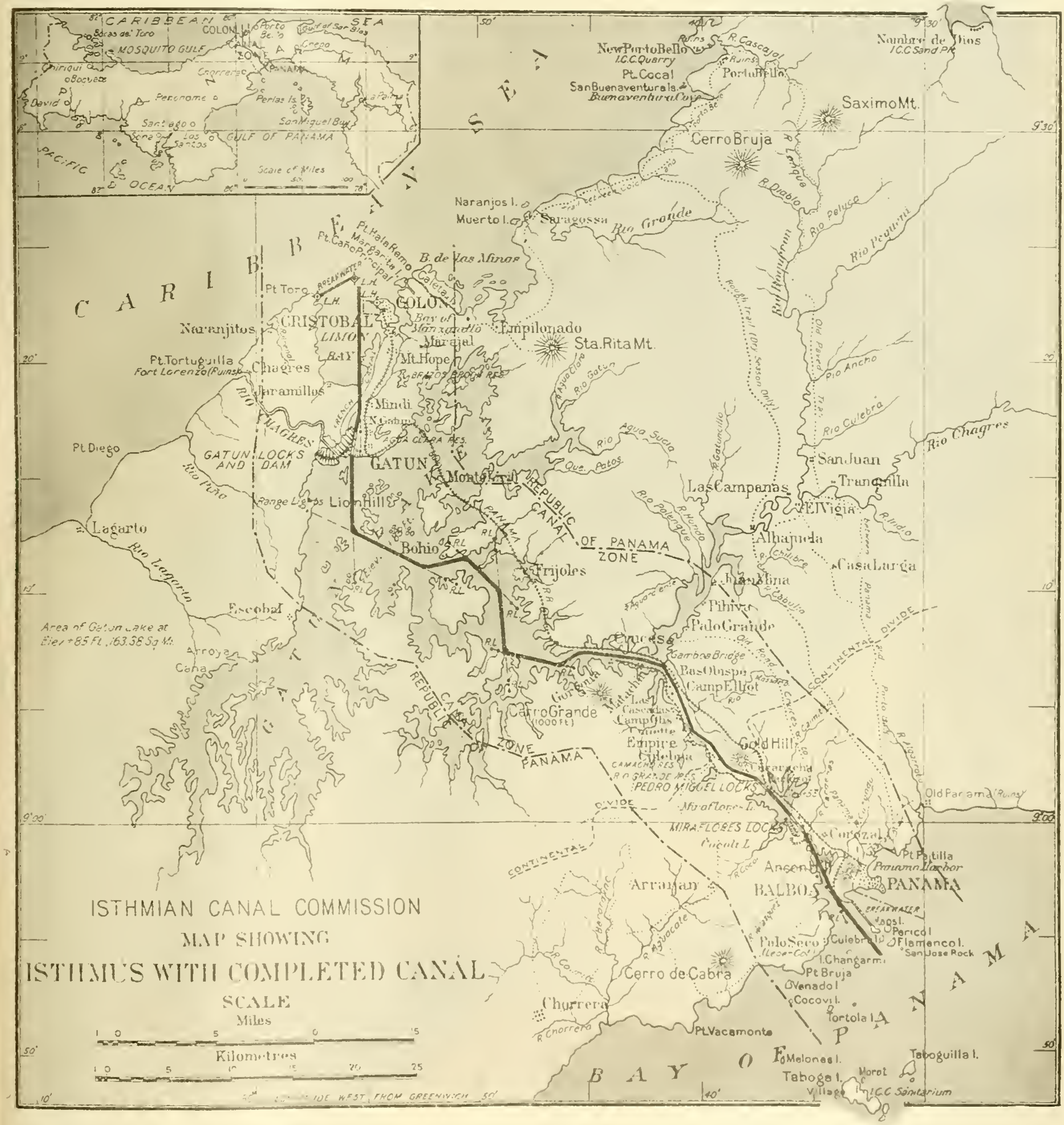


Fig. 2. Map of the Isthmus of Panama Showing the Canal Zone and the Route of the Panama Canal.

t is the opinion of the committee that, if the United States was not the owner of any of these lands, and was obliged to acquire them today, in view of the certainty of the canal's completion, tho cost would not be less than $1,000,000.

The committee considered that the French hannel from Balboa to deep water, which was n use for approximately the first four years of American occupation was worth, to the Commission, the sum of $125,000 a year, or $500,000, this being the only approach for commercial shipping and for the delivery of supplies on he Pacific side during that period.

There was a considerable amount of clearing and roadmaking done by the French at several points in the Canal Zone, notably at Ancon, Cristobal and Empire. The committee consider the sum of $100,000 a moderate estimate of the value of this work.

Recapitulation:

Excavation	$25,389,240
Panama Railroad Stock	9,644,320
Maps, drawings and records	2,000,000
Material and equipment	2,112,063
Buildings	2,054,203
[illegible]	1,000,000
[illegible]	500,000
[illegible]	100,000
Total	$42,799,826

[illegible] of Three Provisional Plans for the Panama Canal.

[illegible] Canal Commission re[illegible] general plans for a lock canal at Panama. The Majority members of the International Board of Consulting Engineers, reporting in 1906, drafted general plans for a sea level canal at Panama. The Minority members of the International Board of Consulting Engineers, reporting in 1906, drafted plans for a lock canal at Panama. The final plan on which the canal has been built is the plan of the Minority members of the International Board of Consulting Engineers as modified by the third Isthmian Canal Commission.

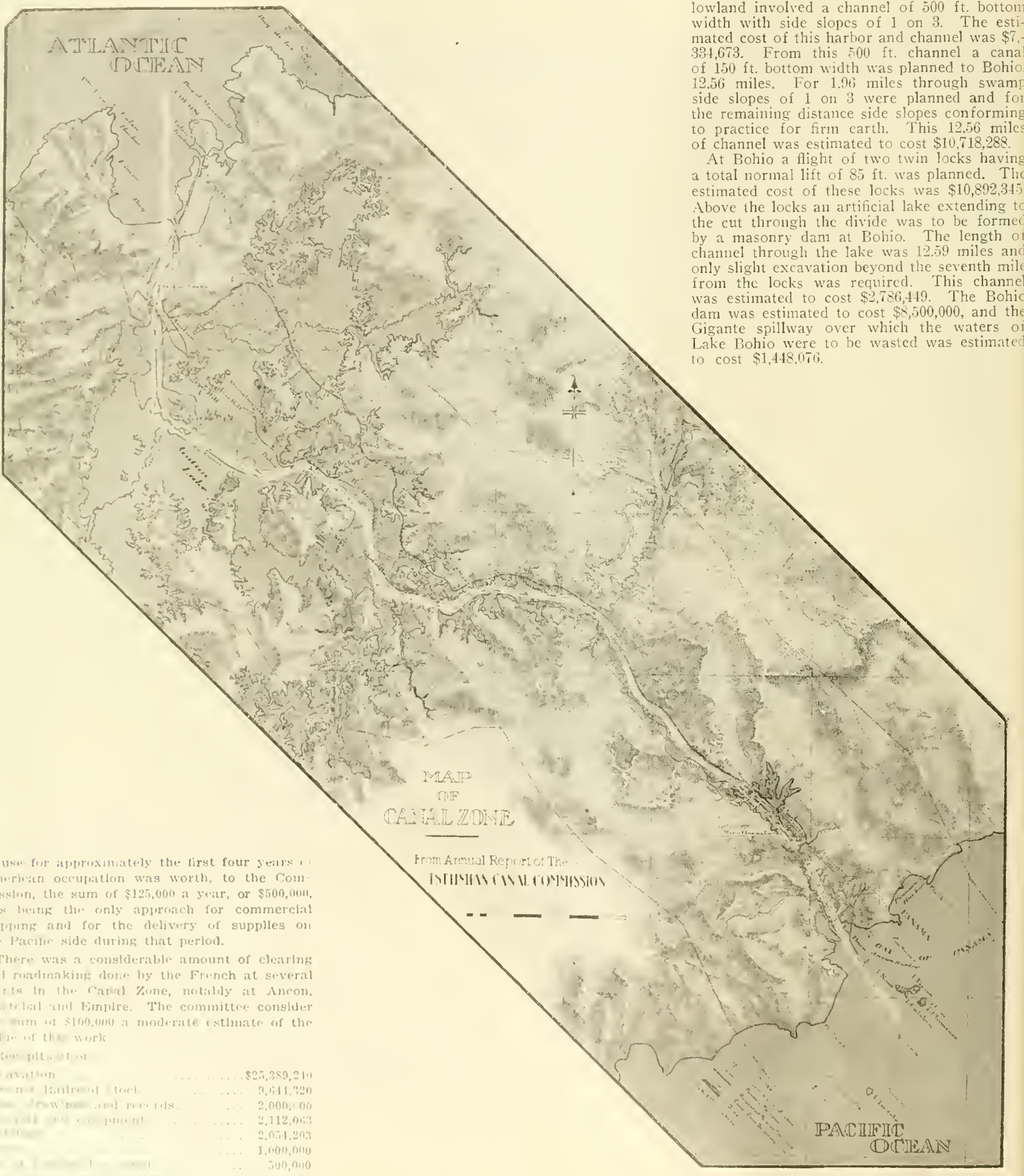


Fig. 3. Topographical Map of the Canal Zone and the Panama Canal.

PLAN OF THE FIRST ISTHMIAN CANAL COMMISSION.

A profile of the Panama canal as planned by the Isthmian Canal Commission in its report of 1901 is given by Fig. 4. According to this plan excavation began at the 6-fathom line in the Bay of Limon and for a distance of 2.62 miles through the bay and bordering lowland involved a channel of 500 ft. bottom width with side slopes of 1 on 3. The estimated cost of this harbor and channel was $7,334,673. From this 500 ft. channel a canal of 150 ft. bottom width was planned to Bohio, 12.56 miles. For 1.96 miles through swamp side slopes of 1 on 3 were planned and for the remaining distance side slopes conforming to practice for firm earth. This 12.56 miles of channel was estimated to cost $10,718,288.

At Bohio a flight of two twin locks having a total normal lift of 85 ft. was planned. The estimated cost of these locks was $10,892,345. Above the locks an artificial lake extending to the cut through the divide was to be formed by a masonry dam at Bohio. The length of channel through the lake was 12.59 miles and only slight excavation beyond the seventh mile from the locks was required. This channel was estimated to cost $2,786,449. The Bohio dam was estimated to cost $8,500,000, and the Gigante spillway over which the waters of Lake Bohio were to be wasted was estimated to cost $1,448,076.

At the end of Lake Bohio and the entrance of the summit cut it was planned to place a pair of gates 100 ft. wide which would close the cut channel against Lake Bohio and permit the cut to be laid dry without reducing the lake level. These gates were known as the Obispo gates and were estimated to cost $295,436. The summit cut extended from the Obispo gates to the Pedro Miguel locks, a distance of 7.95 miles, of which 6.02 miles were designated as heavy cut. The deepest part of the cut was about 5 miles from Obispo and here the canal bottom was 274 ft. below the natural surface. The cut was planned for a bottom width of 150 ft. and side slopes of 1 on 1. The channel through the cut was to be lined with masonry walls on both sides. These walls were to rise to 2 ft. above high water, have inside faces nearly vertical, and at their tops the cut was to be stepped back to form a bench on each side, one bench being planned to carry the tracks of the Panama R. R. The estimated cost of the 7.95 miles of cut was $44,378,335 and the time of construction was estimated to be eight years. The Commission's statement concerning the stability of the sides of the summit cut is interesting in the light of the slides which developed during the construction of the present canal. It was as follows:

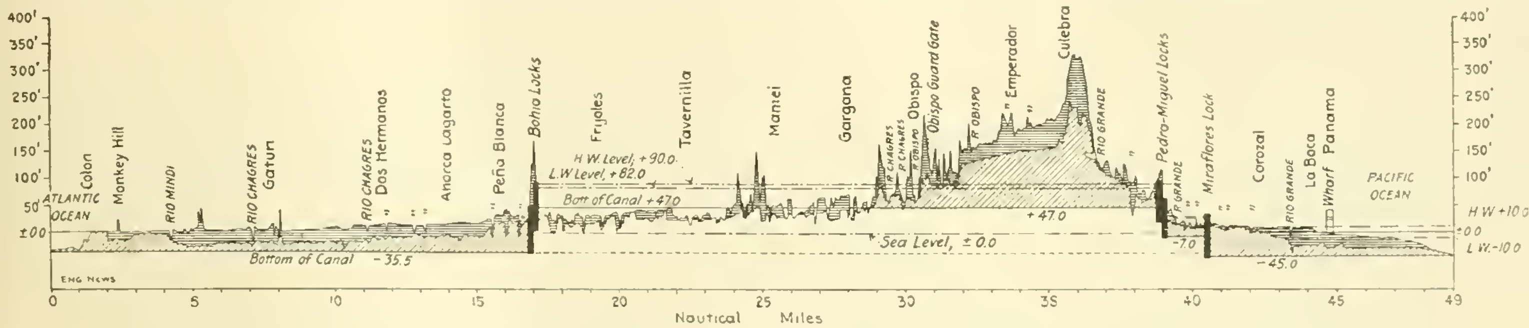


Fig. 4. Profile of the Panama Canal Route Recommended by the Isthmian Canal Commission of 1901.

> Much has been said about the instability of the Culebra cut; in point of fact, there is clay in the upper portion of the deep cut which flows readily when saturated, but which will give little trouble if thoroughly drained; probably nine-tenths of the material would naturally be classified as hard clay of stable character; it would weather somewhat and the surface might require some repairing with concrete in bad places, a practice common with deep cuttings in Europe; this clay disintegrates rapidly in water, and for this reason the canal prism should be confined between masonry walls. With the provision made for broad benches on each side on which any slight slides would be arrested, it is confidently believed that no trouble would be experienced.

At Pedro Miguel a set of two twin locks was planned. Their aggregate lift was calculated to be between 54 and 62 ft. and they were estimated to cost $8,496,326. From Pedro Miguel to Miraflores there would be a level 1.33 miles long controlled by a spillway at Miraflores. This level was estimated to cost $1,169,611. A single twin lock was planned at Miraflores and it, with the spillway, was estimated to cost $5,720,363. From Miraflores a channel 150 ft. wide with side slopes of 1 on 3 was to be excavated to La Boca. This channel, stated to be mostly through low swamp country and to be 4.12 miles long, 200 ft. wide on the bottom and with slopes of 1 on 3, was to extend to the 8-fathom line in Panama Bay. The channel from Miraflores to deep water was estimated to cost $12,366,914.

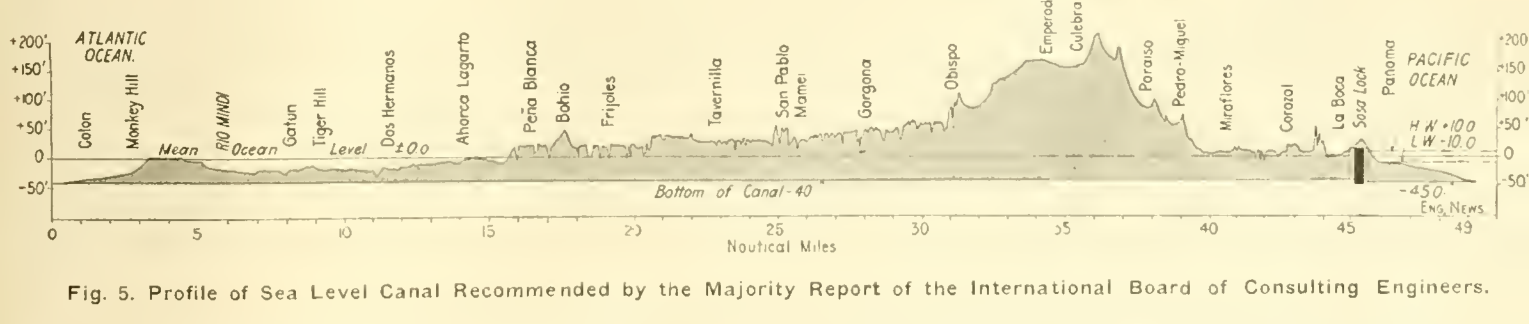


Fig. 5. Profile of Sea Level Canal Recommended by the Majority Report of the International Board of Consulting Engineers.

Besides the canal structures proper the plan involved diversion of the Chagres opposite Gatun, diversion of the Gatuncillo east of Gatun, and reconstruction of the Panama R. R. around Lake Bohio. The estimated costs of these works are given in the recapitulated estimate which follows:

Item.	Cost.
Colon entrance and harbor	$ 7,334,673
Harbor to Bohio locks	10,718,288
Bohio locks	10,982,315
Lake Bohio	2,786,149
Obispo gates	295,436
Culebra sections	44,378,335
Pedro Miguel locks	8,496,326
Pedro Miguel level	1,169,611
Miraflores locks and spillway	5,720,363
Pacific level	12,366,913
Bohio dam	8,500,000
Gigante spillway	1,124,524
Channel between the marshes	1,448,076
Chagres diversion	1,929,976
Gatuncillo diversion	100,000
Panama R. R. reconstruction	1,267,500
Total	$118,618,816
Engineering, police, sanitation, etc.	23,723,763
Grand total	$142,342,579

SEA LEVEL PLAN OF THE INTERNATIONAL BOARD OF CONSULTING ENGINEERS.

Eight of the 13 members of the Board of Consulting Engineers appointed in 1905 to decide on the type of canal at Panama agreed in recommending a sea level canal and presented general plans for such a canal. A profile of the sea level canal recommended is shown by Fig. 5. The route of the canal recommended was in general as follows:

The initial point in the axis of the canal was located in the Bay of Limon at a point about one mile northwest of Manzanillo light where the depth of the water is 40 ft., and the channel was to extend in a straight line to the south end of the bay where it connected by curve at Mindi with the old French canal. The line then followed the partially completed excavation of the French canal to Bohio 12 miles from Mindi. The channel to Bohio first cut the Chagres at Gatun seven miles from Colon and then repeatedly cut it from that point to Bohio. From Bohio to Obispo, 11 miles, the line followed the Chagres, coinciding with or cutting its channel throughout. About three-fourths of a mile from Obispo, at Gamboa, a masonry dam was planned to impound the Chagres, as described in a succeeding paragraph.

Continuing from Obispo the line was through the divide to Pedro Miguel, nine miles. The cut through the divide at the deepest point was 373 ft. deep, measuring from the natural surface. The line from Pedro Miguel on was practically a straight line to deep water in the Gulf of Panama.

From shore to shore the line was 40 mile long; including the harbor channels it wa 49.35 miles long. Of its length 19.17 mile were curve and 30.18 miles were tangent. Th channel was planned for a depth of 40 ft throughout, its width in earth was 150 ft. a the bottom with side slopes, depending on the material, giving surface widths from 302 to 437 ft. In rock the bottom width was 200 ft and the width at water surface was 208 ft At a point between Ancon and Sosa Hill a tidal lock was planned and beyond this lock to deep water in Panama Gulf the channe was 300 ft. wide. At Corozal a set of sluice discharging into the Rio Grande was planned

The sea level canal as planned required control and diversion of the Chagres and it tributaries. As already stated, the main control planned for the Chagres was a dam a Gamboa. A masonry dam rising 130 ft. abov the river bed and 180 ft. above sea level wa proposed. With water at the 170 ft. level th wetted perimeter of the reservoir forme would embrace an area of 29.47 square miles It was figured that the canal prism would without objectionable currents, care for a flov over the Gamboa dam of 15,000 sec. ft. As a double precaution, however, it was planned by building sluices at Corozal to divert half o the flow through the summit cut over the spillway into the Rio Grande on the Pacific side Any flood flow of the Chagres exceeding 15,000 sec. ft. up to 65,000 sec. ft. and of a duration to be expected from records of pas floods was intended to be impounded by th Gamboa Reservoir. The regulation of th tributaries of the lower Chagres was planne to be accomplished by dams and diversion channels.

The estimated cost of excavation of a se level canal as planned was as given by Table 1

TABLE 1.

Item.	Miles.	Quantity.	Cost.
Limon Bay	3.92	2,922,734	$ 624,39
Shore to Mindi	1.57	10,047,473	3,570,08
Mindi to Bohio	11.73	24,557,584	11,557,85
Bohio to Obispo	13.74	60,674,437	43,171,32
Obispo to Pedro Miguel	8.08	109,891,710	95,200,80
Pedro Miguel to Miraflores	1.73	5,429,144	7,711,50
Miraflores to Sosa	4.60	11,330,528	15,103,98
Sosa to Pacific	3.98	5,172,867	6,196,14
Totals	49.35	231,026,477	$183,136,19

Including excavation and all other works he estimated cost was as follows:

Item.	Cost.
Jetties in Limon Bay	$ 5,000,000
Excavation	183,136,107
River diversion	3,500,000
Gamboa dam	6,000,000
Corozal sluices, etc.	920,000
Tidal locks	6,000,000
Lock jetties	795,000
Panama R. R. reconstruction	500,000
Total	$205,851,000
0 per cent engineering, etc.	41,170,200
Grand total	$247,021,200

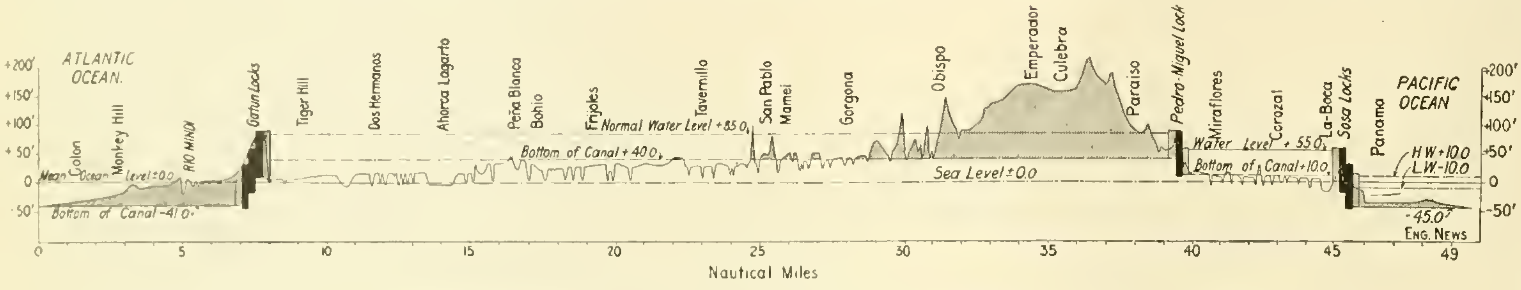


Fig. 6. Profile of Lock Canal Recommended by the Minority Report of the International Board of Consulting Engineers.

The time required for the construction of he canal was estimated at 12 to 13 years. The rgument presented by the majority report in avor of the sea level canal recommended by t was substantially, that it gave safe and ninterrupted navigation to a degree never ossible in a lock canal and that the shorter ime of construction of a lock canal and its maller cost were not of great enough value to ounterbalance this advantage to navigation.

OCK CANAL PLAN OF THE INTERNATIONAL BOARD OF CONSULTING ENGINEERS.

The canal recommended by the minority of ive members of the Board of Consulting Engineers was a lock canal with its summit level t 85 ft. elevation. A profile of the canal is iven by Fig. 6. The general route was subtantially that on which the canal has finally een built and is shown by Figs. 2 and 3.

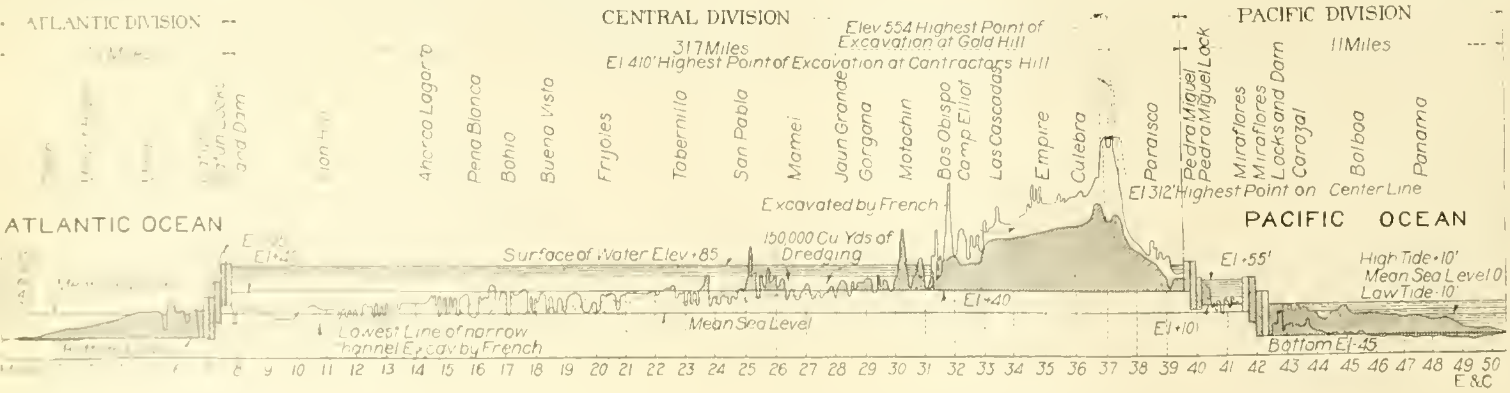


Fig. 7. Profile of Lock Canal as Finally Planned and Constructed.

The sea level section was to extend from leep water in Limon Bay to Gatun, the channel being 500 ft. wide on the bottom. At atun an earth dam was planned across the hagres which should form Lake Gatun exending to Obispo. This inland lake was to erve the double purpose of giving navigaion with very little excavation as far as bispo and of furnishing a large reservoir for eceiving the flood waters of the Chagres. A masonry spillway in the earth dam carried off excess waters and delivered them into the hagres below the dam. To transfer vessels from the sea level channel below Gatun to the 85 ft. summit level formed by Gatun Lake a set of three twin locks having an aggregate ift of 85 ft. was planned.

The channel in Lake Gatun for 15.69 miles was to have a width of 1,000 ft. and a depth of 45 ft. The channel then decreased to 800 ft. for [illegible] miles, then to 500 ft. for 3.73 miles to Obispo. Through the summit the channel was to have a width of 300 ft. from Obispo to Las Cascades, 1.55 miles, and from Las Cascades through the deep cut for 4.7 miles it was to be 200 ft. wide. It then widened to 300 ft. for 1.88 miles to Pedro Miguel. At Pedro Miguel a twin lock with 31 ft. lift was planned.

A set of two twin locks having a total lift of 62 ft. was planned at Sosa. By constructing three dams, one extending from the locks across the Rio Grande to San Juan hill, another from Sosa hill to Ancon hill and a third from Ancon hill in the direction of Corozal to high land, Sosa Lake was to be formed to make the level between Pedro Miguel and Sosa. The channel through this lake was to be 500 ft. wide to 1.64 miles from the Pedro Miguel lock, then 1,000 ft. wide for 3.38 miles to the Sosa locks. From Sosa to deep water in Panama Gulf the channel was to be 300 ft. wide.

The time required to complete the canal as planned was estimated as nine years and this shorter time coupled with the less cost as compared with a sea level canal was considered to be the justification for recommending the lock canal plan. The estimated cost of the canal as planned is given in Table II.

PLAN AS MODIFIED BY THIRD ISTHMIAN CANAL COMMISSION.

The canal plan on which construction was definitely started in 1906-7 was the plan of the minority members of the Board of Consulting Engineers. Two important changes in this plan were found advisable. When the plan was approved in 1906, the Secretary of War raised question of the wisdom of locating the main locks on the Pacific side in so exposed a place as was Sosa Hill and pointed out that only the less cost of this location excused its selection from the military viewpoint. Construction was, however, begun on one of the dams designed to impound Sosa Lake. In a short time indications appeared that the foundation soil was not as stable as the borings had indicated. Further examination was made of the ground showing an unstable soil overlying rock some 10 to 70 ft. deep. New plans were made to fit the newly discovered conditions and cost estimates based on these new plans raised the original estimate of $1,311,000 to $11,574,000. In view of this increase, search was undertaken for a more favorable location. The studies made indicated the existence of rock foundations suitable for one lock at Pedro Miguel and for two locks at Miraflores.

Amended plans were prepared. They provided for a twin lock with 31⅓-ft. lift at Pedro Miguel and for two twin locks with an aggregate lift of 54⅔-ft. at Miraflores. From Miraflores to deep water in Panama Gulf a dredged channel 500 ft. wide and 45 ft. deep was provided. The large Sosa Lake was reduced to a small lake about a mile square between Pedro Miguel and Miraflores. The new plan was recommended for adoption by the Commission and on Dec. 20, 1907, it was approved by President Roosevelt.

In 1908 the Isthmian Canal Commission recommended the building of larger lock chambers and the widening of the channel through Culebra Cut. The first change was authorized on Feb. 15, 1908, and the second on Oct. 23, 1908. The new lock plans called for chambers 110 ft. instead of 100 ft. wide. This added some $5,806,000 to the cost of the locks. The original plan called for a channel 200 ft. wide through Culebra Cut; the new plans increased the width to 300 ft.

These changes naturally increased the probable cost of the canal and because of them and for other reasons a new estimate of cost was made in 1908 by the Isthmian Canal Commission. This estimate compared with that of 1906 is given in detail in Table II. In explanation of this estimate Chief Engineer Goethals in his report for 1909 said:

> The act of June 28, 1902, authorized the construction of the canal and made provisions for the necessary funds, the amount stipulated being based on the report of the board of engineers in its report of 1899-1901, and aggregating $144,233,358, including sanitation and police. A modification of the plans then submitted was made by the minority of the board of consulting engineers and adopted by Congress in 1906, which fixed the cost for engineering and construction, exclusive of the purchase price, the cost of sanitation and civil government and the interest, at $139,705,200. In the preparation of this estimate, the same unit prices were assumed as were adopted by the Board of 1899-1901. In the interval, however, wages had increased, and the cost of materials and manufactured articles had risen. Prosperous conditions in the United States, combined with the unsavory reputation that the Isthmus had re-

TABLE II.—ESTIMATES OF COST OF PANAMA CANAL BY THE MINORITY BOARD OF CONSULTING ENGINEERS OF 1906 AND BY THE ISTHMIAN CANAL COMMISSION OF 1908.

ATLANTIC DIVISION.

1906.

	Cu. yds.	Unit.	Amount.	
(1) Breakwater in Limon Bay:				
From M. O. to M. 3.30 as per estimate of the board	....	.....	$5,000,000	
Extension from M. 3.30 to head of bay (M. 4.45)	400,000	$0.75	300,000	
				$5,300,000

The breakwater contemplated by the "Board of 1906" consisted of:	Miles.
An east jetty extending from deep water to Manzanilla Point	0.95
A west jetty extending from deep water to the head of Limon Bay, practically parallel to the axis of the canal	4.55
Total	5.50

No quantities were given by the board for the first 3.30 miles.

The difference in money between the estimate of the board of 1906 and the present estimate is largely because of the difference in unit prices.

1908.

	Cu. yds.	Unit.	Amount.	
1) Breakwater in Limon Bay	.4,234,000	$2.70	$11,432,000	
				$11,432,00

	Miles
The breakwater upon which this estimate is made consists of a west jetty with outer end in 44 feet of water and trending toward Toro Point	1.9
And an east jetty from 44 feet of water and extending eastwardly parallel to a line drawn between the lighthouses at Manzanilla and Toro Point	1.0
Total	2.9

The estimate made in 1906 by the Board of Consulting Engineers was based upon a breakwater, the foundations largely to be of material taken from the canal prism.

The estimate of 1908 is based upon all material for the breakwater being obtained from Porto Bello.

By the change in location the present excavation from the canal can not be used.

1906.

	Cu. yds.	Unit.	Amount.	
(2) Channel from M. O. to Gatun locks (500 feet wide):				
Dredging M. to M. 4.55—				
Mud and sand	8,300,000	$0.15	$1,245,000	
Dredging M. 4.55 to Gatun locks—				
Soft material, suction dredges	10,000,000	.25	2,500,000	
Indurated clay, suction dredges with cutters	2,030,000	.70	1,421,000	
	20,330,000	$0.254+	....	$5,166,000

1908.

	Cu. yds.	Unit.	Amount.	
(2) Channel from M. O. to Gatun locks (500 feet wide):				
Dredging—				
Mud and sand	13,260,682	$0.20	$2,652,000	
Earth requiring cutting and some blasting	10,648,137	.45	4,792,000	
Rock blasted and dredged	1,105,313	1.50	1,658,000	
Rock under water	851,267	2.50	2,128,000	
Excavation—				
Earth in dry	152,234	.45	69,000	
Earth in dry, blasted	608,895	.60	365,000	
Rock	3,078,596	1.55	4,772,000	
	29,705,124	$0.553+	....	$16,436,00
Maintenance of channel after July 1, 1909	6,500,000	.20	....	1,300,00
	36,205,124	$0.456+	....	$17,736,00

Excess of present estimate over board of 1906		$12,570,000
By reason of maintenance	$1,300,000	
By reason of increase in quantities, 9,375,124 cu. yds.	4,882,000	
By increase in unit prices due largely to rock, 20,330,000	6,388,000	
Total	$12,570,000	

1906.

	Cu. yds.	Unit.	Amount.	
(3) Gatun locks: Flight of 3 twin locks, 95 by 900 feet usable dimensions; total lift 85 feet:				
Excavation, earth	920,000	$0.40	$ 368,000	
Excavation, indurated clay (rock)	2,740,000	.70	1,918,000	
	3,660,000	$0.60	....	$2,286,000
Locks, exclusive of excavation and approach walls			$13,075,000	
Emergency gate			1,000,000	
Approach walls			500,000	
				14,575,000
Back fill and embankment	660,000	.50	....	330,000
				$17,191,000

1908.

	Cu. yds.	Unit.	Amount.	
(3) Gatun locks: Flight of 3 twin locks, 110 by 1,000 feet usable dimensions; total lift 85 feet:				
Excavation—				
Earth	837,667	$0.35	$ 293,000	
Rock (indurated clay)	2,948,237	.65	2,211,000	
Rock, by hand, preparing foundations	100,000	2.00	200,000	
Excavation, rock for curtain walls	103,400	5.50	569,000	
Excavation, dredging	1,150,000	.33	280,000	
Total excavation	5,139,304	69.1+	....	$3,553,00
Concrete, including approach walls	2,096,000	7.75	16,244,000	
Emergency dam, gates, valves, operating machinery, electric capstans, etc.			5,220,000	
				21,464,00
Back fill	949,000	.85	....	807,00
				$25,824,00

Present estimate exceeds estimate of 1906 by	$ 8,633,000	
The following are:		
Due to increased size of locks and change in plans—		
Excavation—		
Curtain walls, 103,400 cubic yards, at 5.50 cents	569,000	
Other excavation, 1,375,904 cubic yards, at 50.7 cents	698,000	
Total		$1,267,000
Concrete—		
Present estimate, 2,096,000 cubic yards, at $7.75	16,244,000	
Report of 1906, 1,300,000 cubic yards, at $8	10,400,000	
Increase		5,844,000
Gates, operating machinery, etc.—		
Present estimate	5,220,000	
1906 estimate	4,175,000	
Increase		1,045,000
Due principally to furnishing 6 intermediate lock gates		
Back fill		477,000
Total		$8,633,000

1906.

	Cu. yds.	Unit.	Amount.	
(4) Gatun dam:				
Earth embankment—				
Below elevation 50, dredged material pumped into dam	12,170,000	$0.20	$2,434,000	
Between elevation 50 and elevation 80, dredged material pumped into dam	5,770,000	.30	1,731,000	
Above elevation 80, material for excavation placed in dam	3,260,000	.25	815,000	
	21,200,000	234+	...	$4,980,000
Excavation for diversion channel and regulating works (spillway)—				
Earth	1,100,000	.40	440,000	
Indurated clay (rock)	480,000	.70	336,000	
	1,580,000	.491+	....	776,000
Concrete in regulating works and diversion channel	189,000	8.00	1,512,000	

1908.

	Cu. yds.	Unit.	Amount.	
(4) Gatun dam.				
Earth embankment—				
Dredged material pumped into dam	7,000,000	$0.20	$1,400,000	
Dredged material pumped into dam	7,000,000	30	2,100,000	
Dredged material pumped into dam	7,000,000	45	3,150,000	
Rock fill	3,400,000	20	680,000	
	24,400,000	.30	....	$7,330,0
Excavation for spillway, etc.	1,312,882	70	919,000	
Excavation for sheet piling and stripping	360,000	1.20	432,000	
Excavation for curtain walls of regulating works	60,000	2.10	126,000	
	1,732,882	.852+	....	1,477,0
Concrete	200,000	8.25	1,650,000	
Regulating gates and machinery	.....	.....	320,000	

	Cu. yds.	Unit.	Amount.	
Regulating gates and machinery			320,000	
				1,332,000
Preparatory and miscellaneous				200,000
				$7,788,000

				1,970,000
Sodding				100,000
Sheet piling, linear feet	7,000	60.00		420,000
Riprap	700,000	2.00		1,400,000
Miscellaneous (includes filling, trestles, pumping out old channels, etc.)				875,000
				$13,572,000

Present estimate exceeds estimate of 1906 by $5,784,000, as follows:

Embankment:		
Due to rock fill (new item), 3,400,000 cubic yards, at 20 cents	$ 680,000	
Due to increase in unit prices	1,670,000	
		$2,350,000
Spillway excavation:		
For curtain walls (new item), 60,000 cubic yards, at $2.10	126,000	
For other excavation, 92,882 cubic yards	575,000	
		701,000
Concrete		138,000
New items:		
Sodding, sheet piling, riprap, etc		2,595,000
Total		$5,784,000

Breakwater	$ 5,300,000	Breakwater	$11,432,000
Channel M. O. to Gatun locks	5,166,000	Channel M. O. to Gatun locks	17,736,000
Gatun locks	17,191,000	Gatun locks	25,824,000
Gatun dam	7,788,000	Gatun dam	13,572,000
	$35,445,000		$68,664,000

CENTRAL DIVISION.

(5) Channel from Gatun locks to M. 30.6: 15.42 M. 1,000 feet wide, 3.6 M. to 800 feet wide, 3.6 M. 500 feet wide:

Excavation—	Cu. yds.	Unit.	Amount.	
Earth, dry	600,000	$0.40	$ 240,000	
Indurated clay (rock)	130,000	.70	91,000	
Rock, in the dry	1,160,000	1.15	1,334,000	
Dredging	12,960,000	.25	3,240,000	
				$4,905,000
Cutting trees in Gatun Lake				100,000
	14,850,000	.337		5,005,000

(6) M. 30.6 near Bas Obispo to M. 39.0 Pedro Miguel lock: 0.65 M. 500 feet wide, 3.05 M. 300 feet wide, 4.7 M. 200 feet wide:

	Cu. yds.	Unit.	Amount.	
Excavation, dry	53,765,000	.80	43,012,000	
Allowance for diversion channels between Bas Obispo and Pedro Miguel			850,000	
				43,862,000
Total central division				48,867,000
Total excavation	68,615,000	.712		48,867,000

(5) Channel from Gatun locks to M. 30.6: 15.42 M. 1,000 feet wide, 3.0 M. 800 feet wide, 0.50 M. 700 feet wide, 3.6 M. 500 feet wide:

Excavation—	Cu. yds.	Unit.	Amount.	
Earth and rock, dry	7,752,198	$0.90	$6,977,000	
Earth, dredging	4,000,000	.25	1,000,000	
	11,752,198	.6887		$7,977,000

(6) M. 30.6 near Bas Obispo to M. 39.0 Pedro Miguel locks: 0.65 M. 500 feet wide, 7.75 M. 300 feet wide:

	Cu. yds.	Unit.	Amount.	
Excavation, dry	78,042,295	.98	76,481,000	
Concrete	400,000	10.00	4,000,000	
				80,481,000
Total central division				88,458,000
Total excavation	89,794,493	.94		84,458,000
Concrete	400,000	10.00		4,000,000

The present estimate is greater than that of the Board of 1906.

By reason of increase in quantity, 21,179,493 cubic yards, at 94 cents	$19,921,000	
By reason of increase in unit prices, 68,615,000 cubic yards, at 24 1/10 cents	16,520,000	
		$36,441,000
By reason of concrete revetment, 400,000 cubic yards, at $10		4,000,000
		40,441,000
Less lump sum for diversion in 1906 estimate		850,000
		$39,591,000

In the 1906 estimate the cost of concrete revetment was included in the 20 per cent allowed for administration, contingencies, etc.

Of the increased yardage, 15,000,000 cubic yards is due to widening canal from 200 to 300 feet for 4.70 miles.

PACIFIC DIVISION.

(7) Pedro Miguel Lock: 1 twin lock 95 by 900 feet usable dimensions, 30-foot lift:

Excavation—	Cu. yds.	Unit.	Amount.
Earth	650,000	$0.40	$ 260,000
Rock	520,000	1.15	598,000
	1,170,000		858,000
Back filling	390,000	.50	195,000
Locks exclusive of excavation and approach walls			5,935,000
Emergency gate			1,000,000
Approach walls			300,000
			$8,288,000

(8) Pedro Miguel Dam:

	Cu. yds.	Unit.	Amount.
Earth embankment	1,100,000	.25	275,000
Preparatory work			50,000
			$ 325,000

(7) Pedro Miguel Lock: 1 twin lock 110 by 1,000 feet usable dimensions, 30-foot lift:

	Cu. yds.	Unit.	Amount.
Excavation	968,901	$1.15	$1,114,000
Do	299,000	1.30	389,000
	1,267,901	1.18	1,603,000
Back filling	834,500	.55	459,000
Concrete	922,600	8.25	7,611,000
Emergency dam, gates, valves, operating mechanism, electric capstan, etc			3,100,000
Miscellaneous			20,000
			$12,693,000

(8) Pedro Miguel Dam:

	Cu. yds.	Unit.	Amount.
Excavation and puddle core	21,200	1.20	25,000
Embankment, earth	230,500	.95	219,000
Miscellaneous			7,000
			$ 251,000

The increase of the present estimate for Pedro Miguel locks over the estimate of 1906 is due to increase in size of locks and changes in plans and amounts to	$4,405,000
As follows:	
Increase due to excavation, 97,901 cubic yards	645,000
Increase due to back fill, 444,000 cubic yards	264,000
Increase due to concrete due to increase in size of locks, 224,000 cubic yards, at $8.25	1,848,000
Due to change of plans, 100,000 cubic yards, at $8.25	825,000
Increase due to gates, operating machinery, etc., including intermediate gates added	823,000
	$4,405,000

NEW PANAMA RAILROAD.

(12) New Panama R. R.:

Gatun to Bohio	$2,000,000
Bohio to Pedro Miguel	1,300,000
Pedro Miguel to Panama	400,000

(12) New Panama R. R.:

Before October 1, 1908			$1,783,000
October 1, 1908, to July 1, 1909			1,020,000
After July 1, 1909—			
Engineering		$ 90,000	
Real estate and right of way		20,000	
Grading—			
Earth excavation, 330,000 cubic yards, at $0.40	$ 132,000		
Rock excavation, 405,500 cubic yards, at $1	405,500		
Borrow, 4,800,500 cubic yards, at $0.40	1,920,200		
		2,457,700	
Bridges and culverts, 63,960 cubic yards concrete, at $10		639,600	
Temporary trestles and structures		395,000	
Temporary track, 48,720 feet, at $2		97,440	

$3,700,000

Permanent track—			
35 miles ties (spaced 21 inches), 105,600 ties, at $1.25	132,000		
70 miles 70-pound rail and fastenings, at $1 per foot of track	184,800		
35 miles of ballast, at $4,752 per mile	166,320		
35 miles of track laying and surfacing, at 50 cents per foot	92,400		
		575,520	
Fencing		30,000	
Interlocking and signals		30,000	
Telegraph and telephone lines		50,000	
Station buildings and section houses		75,000	
Water stations		10,000	
Fuel stations		20,000	
Miscellaneous structures		10,000	
General expenses		75,000	
Tools		120,000	
			4,695,26
Maintenance—			
37 miles for two years, at $9,000 per mile per annum			666,00
			$8,164,00

LAND DAMAGES.

(13) Land damages:
Land damages in Gatun and Sosa lakes .... $ 300,000

(13) Land damages:
Land damages in Gatun Lake .... $ 500,00

GENERAL ITEMS.

Municipal improvements:

Expenditures under this head include everything for the construction, operation, maintenance, and repair of roads, walks, waterworks pumping plant, reservoirs, sewers, special grading, etc., required for canal construction work proper in the Canal Zone—i. e., for its employes and also for the above work performed in the cities of Panama and Colon for sanitation under Article VIII of the treaty.
United States Treasury by water rates collected by the United States. For further improvements in Panama and Colon for sanitation a supple

These expenditures in and for the cities of Panama and Colon have amounted to about $2,500,000 to date, and this sum will be returned to th
mentary estimate of $1,200,000 was forwarded to Congress in December, 1908, which, if appropriated, will likewise be returned to the United State
Treasury.

Expenditures for these purposes in the Canal Zone for the benefit of native villages and those not employed by the commission are not pai from congressional appropriations but from Canal Zone funds, derived from taxes collected by the district tax collectors, and no allowance fo same is made in figures given below.

Expenditures to October 1, 1908		$5,982,000
Estimates from October 1, 1908, to July 1, 1909		700,000
Estimates after July 1, 1909:		
Atlantic division—		
Construction	$585,000	
Maintenance	575,000	
		1,160,000
Central division		1,800,000
Pacific division		822,000
Quartermaster's department		450,000
Cities of Panama and Colon—grading, paving, waterworks, sewers, and sanitation		1,200,000
		$12,114,000

(9) Channel Pedro Miguel to deep water 1.87 M., 500 feet wide, Pedro Miguel to New Miraflores, 3.61 M., not less than 1,000 feet wide in Sosa Lake; 4.23 M., 300 feet wide in Panama Bay:

Pedro Miguel to La Boca—			
Excavation—dredging	1,680,000	$0.25	$ 420,000
La Boca to deep water—			
Excavation—			
Dredging	4,760,000	.15	714,000
Rock under water	200,000	1.50	300,000
Rock under water	370,000	2.50	925,000
	$7,010,000	$0.33+	2,359,000

(9) Channel Pedro Miguel to deep water, 500 feet wide:

Excavation—			
Dredging	32,763,051	$0.20	$6,553,00
Rock under water	1,630,000	2.50	4,075,00
Dry	2,141,548	1.00	2,142,00
	$36,534,599	$0.35	$12,770,00
Removing wrecks			100,09
Breakwater to Island Naos			300,00
			$13,170,00

From Pedro Miguel locks to the Pacific the estimates of the board of 1906 are not comparable item by item with present estimates on account of change in plan.

The increased quantity of work itemized above is due to extending the sea-level channel from La Boca to Miraflores, and to shifting the position of the channel between La Boca and deep water.

Pacific locks and dams.

(10) La Boca locks—flight of two twin locks, 95 by 900 feet, usable length—total maximum lift 64.7 feet:

Excavation—			
Earth	480,000	$0.40	$ 192,000
Rock	950,000	1.50	1,425,000
	$1,430,000	$1.13—	$1,617,000
Back fill and embankment			475,000
Locks, exclusive of excavation and approach walls			11,000,000
Approach walls			450,000
			$13,542,000

Pacific locks and dams.

(10) Miraflores locks—flight of two twin locks, 110 by 1,000 feet, usable length—total maximum lift 64.7 feet:

Excavation—			
Dry	1,132,587	$1.16	$1,246,00
Dry rock	258,880	1.50	388,00
Dry rock	845,545	2.25	1,902,00
Dredging	1,471,160	.25	368,00
	$3,708,172	$1.05+	$3,904,00
Back fill	1,922,400	.45	865,00
Piling, linear feet	124,900	.50	62,00
Riprap	43,500	.40	17,00
Concrete	69,500	6.50	452,00
Concrete	1,258,800	8.25	10,385,00
Emergency dam, gates, valves, operating machinery, electric capstans, etc			3,700,00
Miscellaneous			330,00
			$19,715,00

(11) Dams:

Embankment, Ancon-Sosa and Ancon-Corozal dams	5,980,000	.25	1,495,000
Embankment, La Boca dam	6,300,000	.25	1,575,000
Shutting off the tidal flow at La Boca			100,000
Diversion channel and regulating works between Sosa and Ancon			275,000
Preparation of sites, etc			150,000
			$3,595,000

(11) Miraflores dams:

Excavation	184,686	1.00	185,00
Filling	1,051,500	.50	526,00
Concrete	68,500	11.00	754,00
Regulating works			325,00
Miscellaneous			366,00
			$2,156,00

The present estimate for locks at Miraflores exceeds the estimate of 1906 for lock at La Boca $6,173,000

The increase is due principally to increased work from changing size of locks from 95 feet by 900 feet to 110 feet by 1,000 feet, and also to changes in plans, such as providing intermediate gates, etc., viz.:

Increased excavation, 2,278,172 cubic yards	$2,287,000
Increased back-fill	390,000
Piling, riprap, and miscellaneous (new items)	409,000
Increased concrete due to deeper foundation required at Miraflores, 69,500 cubic yds., at $6.50	452,000
Increased concrete due to change in plans, 240,000 cubic yards, at $8.25	1,980,000
Increase due to gates, operating machinery, etc	655,000
Total	$6,173,000

The estimates of 1906 for the Pacific dams as given above is not comparable with the estimate of December, 1908, for Miraflores dams, as the work is not in the same location, and dams, as proposed by the board of 1906, if built, would have had to have been modified in plan.

SUMMARY.

	Cu. yds.	Unit.	Amount.
Pedro Miguel lock	......	.....	$8,288,000
Pedro Miguel dam	......	.....	325,000
Channel Pedro Miguel to deep water	......	.....	2,359,000
Pacific locks (La Boca)	......	.....	13,542,000
Pacific dams (La Boca)	......	.....	3,595,000
			$28,109,000

	Cu. yds.	Unit.	Amount.
Pedro Miguel lock	......	.....	$12,693,000
Pedro Miguel dam	......	.....	251,000
Channel Pedro Miguel to deep water	......	.....	13,170,000
Pacific locks (Miraflores)	......	.....	19,715,000
Pacific dams (Miraflores)	......	.....	2,156,000
			$47,985,000

CONSTRUCTION AND REPAIR OF BUILDINGS FOR THE DEPARTMENT OF CONSTRUCTION AND ENGINEERING.

Expenditures to October, 1908 (including over $1,000,000 for departments of civil administration and sanitation, for which department of construction and engineering has not yet been credited)	$9,951,000
Estimate from October 1, 1908, to July 1, 1909	900,000
Estimate after July 1, 1909:	
Atlantic division	535,000
Central division	965,000
Pacific division	1,000,000
Quartermaster's department	1,300,000
	$14,651,000

Of the above amount, about $10,000,000 is for new construction, and the remainder being for improvements, maintenance, and repair to old French and new American buildings.

About 3,500 buildings of all classes are covered by the estimate. The above expenditures up to October 1, 1908, include over $1,000,000 for the departments of sanitation and civil administration prior to July 1, 1907, for which the appropriations for the department of construction and engineering can not be credited without special provision by law.

For expenditures after July 1, 1909, amounting to $3,800,000, as above, it has been assumed that work under the head of "Repairs" will amount to 5 per cent per annum on the first cost of the buildings to take care of deterioration of paint, woodwork, screening, etc., due largely to climatic conditions. The balance is inserted as a contingent item to provide for any new construction that may be found necessary.

GENERAL EXPENSES.

This includes all general expenses and overhead charges of the commission not properly chargeable directly to any one item of construction work, and includes the following:

All expenses in the United States, all expenditures in the office of the chairman and chief engineer, office of the examiner of accounts, and the disbursing officer. It includes certain expenses of the subsistence department borne by the commission, expenditures for telegraph and telephone service, transportation on the Isthmus, meteorological work, Young Men's Christian Association work, etc.

Expenditures to October 1, 1908	$4,980,000
After October 1, 1908, 6¼ years, at $3,000,000	18,750,000
	$23,730,000

The estimate of $3,000,000 a year, or $250,000 a month, is based on the following:

Office chairman and chief engineer	20,000
Office secretary of commission	2,000
Engineering and technical force, attached to office of chief engineer	9,000
Meteorological and river hydraulic work, experimental work, and general surveys	6,000
Salaries of Y. M. C. A. secretaries and expenses in connection therewith	3,000
Quartermaster's department	125,000
Office examiner of accounts	20,000
Office disbursing officer	7,500
Proportion of expenses of subsistence department	8,500
All expenses in the United States, including those of Washington office and elsewhere	27,000
Transportation on the Isthmus (paid to Panama Railroad)	15,000
Telephone and telegraph service (paid to Panama Railroad)	7,000
Total	$ 250,000

LOANS TO PANAMA RAILROAD.

This item includes appropriations to date, amounting to about $4,200,000 for the reconstruction and re-equipment of the Panama Railroad, which has been expended for the purchase of new equipment and other improvements properly classed as "betterments and renewals," and for which the resources of the railroad were insufficient.

In accordance with the appropriation acts the repayment of the above sum to the United States has been secured by notes bearing 4 per cent interest, deposited in the United States Treasury.

It is estimated that additional appropriations amounting to $4,100,000 will be required for similar purposes as a loan to the Panama Railroad will be required before the completion of the canal, for improved and enlarged dock and wharf facilities at La Boca and Cristobal, viz:

Expenditures to July 1, 1909, for reconstruction and re-equipment of Panama Railroad		$4,200,000
Estimate for year 1909-10:		
Construction and extension of docks	$ 700,000	
Year 1910-11:		
Reconstruction and extension of docks	1,400,000	
Year 1911-12:		
New coal-handling plant and reconstruction of docks	2,000,000	4,100,000
		$8,300,000

GENERAL SUMMARY.

Board of 1906.

Atlantic division	$35,445,000	
Central division	48,867,000	
Pacific division	28,109,000	
New Panama Railroad	3,700,000	
Land damages	300,000	$116,421,000
20 per cent for administration, engineering, and contingencies, but not including interest during construction, sanitation, and expenses of Zone government		23,284,200
Cost of canal, as estimated by the board of 1906, exclusive of interest during construction, sanitation, and expenses of Zone government		$139,705,200

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garding its healthfulness, made it necessary, in order to secure labor, to increase the wage scales from [illegible] to 60 per cent over those paid in the United States for similar classes of work. Certain gratuities as additional inducements were also offered, which in the main have since been continued. Moreover, the provisions of the eight-hour law were made applicable to the Isthmus.

The unit prices were therefore materially increased. Various changes had also been made in the adopted plan, increasing the quantity of work to be done. In addition more detailed surveys [illegible] were originally available had been made and the work had progressed sufficiently to secure reliable data relative to unit costs. This enabled a revised estimate to be prepared [illegible] at a hearing before the Committee on Appropriations of the House of Representatives in February, 1909.

1908.

Atlantic division	$68,564,000	
Central division	88,458,000	
Pacific division	47,985,000	
New Panama Railroad	8,164,000	
Land damages	500,000	$213,671,000
General items, corresponding to 20 per cent for administration, etc., of minority estimate:		
Light-houses, buoys, and beacons	$ 500,000	
Roads, waterworks, sewers, etc., termed "Municipal improvements" (as per itemized statement)	12,114,000	
Construction and repairing buildings for the department of construction and engineering (as per itemized statement)	14,651,000	
Double tracking Panama Railroad	1,200,000	
Purchase of steamers Panama and Colon (leased to Panama Railroad)	1,300,000	
Purchase of steamers Tremont and Shawmut	1,550,000	
Docks and wharves leased to Panama Railroad Co.	500,000	
Purchase of land not flooded by Gatun Lake	100,000	
General expenses (as per itemized statement)	23,730,000	
Purchase of Panama Railroad stock	150,000	
Contingencies, including fire and marine risks	20,000,000	
Loans to Panama Railroad Co. as per itemized statement)	8,300,000	84,095,000
		$297,766,000
Sanitation:		
Expenditures to October 1, 1908	$ 7,553,000	
Estimate after October 1, 1908, 6¼ years at $2,000,000	12,500,000	$20,053,000
Civil administration:		
Expenditures to October 1, 1908	2,382,000	
Estimate after October 1, 1908, 6¼ years, at $800,000	5,000,000	7,382,000
Payments for franchises, as authorized by the Spooner Act of June 28, 1902:		
Amount paid to New French Canal Company	40,000,000	
Amount paid to Republic of Panama	10,000,000	50,000,000
		$77,435,000

The following is an estimate of reimbursements to the United States Treasury from the foregoing appropriations, not including any allowance for salvage from plant upon completion of the work:

Municipal improvements, cities of Panama and Colon	$ 3,700,000
Loans to Panama Railroad Company	8,300,000
Miscellaneous receipts returned to the United States Treasury, not credited to appropriations for the canal	3,000,000
	$15,000,000

This estimate shows that nearly 50 per cent more work is necessary in order to complete the canal than was contemplated by the original estimate and that the unit prices, due to labor conditions, cost of materials, and gratuities given the employes, have been increased about 20 per cent. The estimate, as prepared, shows that the total cost of engineering and construction alone sums up to $297,766,000; to which, if the purchase price and the estimated cost of sanitation and civil government be added, there results the sum of $375,201,000 as the total cost of the canal.

The Final Panama Canal Plans.

The development of tentative plans toward a final plan for the Panama Canal has been outlined. A general description of the canal constructed now follows, preceding more particular descriptions of the individual works which together form the new waterway.

Beginning at the Atlantic end the first 4½ miles of the canal lie within the Bay of Limon and consist of a channel 500 ft. wide on the bottom, 41 ft. deep and with side slopes of 1 on 3. The alignment of this submerged channel is straight and this tangent continues inland one mile farther or to the Mindi Hills. Here by a curve of long radius the canal swings slightly west and continues on tangent 1½ miles to Gatun. The 2½ miles of inshore channel is of the same dimensions as the submerged channel in Limon Bay. The total canal distance from the terminus in Limon Bay to Gatun is 7 miles and this is the Atlantic sea level of the canal.

At Gatun the Chagres River is dammed by an earth dam having a crest elevation of +105 ft. or 105 ft. above mean sea level. This dam impounds the Chagres, forming Gatun Lake, having a water level normally at +85 ft. elevation but varying from 5 ft. below this level to 2 ft. above it. To reach this 85 ft. level of Lake Gatun there is a flight of three twin locks or three pairs of locks continuous with each other from end to end. The lift of each pair of locks is 28⅓ ft., or for the three pairs a total of 85 ft. The two locks of each pair are side by side, with only a dividing wall between, but this dividing wall is continuous throughout the length of the three locks. The locks at Gatun consist of two contiguous parallel flights of three locks which can be used simultaneously or separately. The chamber of each lock is 1,000 ft. long and 110 ft. wide. Vessels are towed through the locks by electric locomotives.

Lake Gatun, with normal water level at elevation +85 ft., extends in the direction of the canal line as far as Bas Obispo and except for the removal of the top of an occasional mound furnishes a channel without excavation for 17 miles. The line of the channel bends slightly to the east on leaving the Gatun locks and for 3½ miles is 1,000 ft. wide and 75 ft. deep; it then swings more sharply east and continues on tangent to near Bohio, a distance of 4½ miles. This 4½ miles of channel is 1,000 ft. wide with a depth gradually decreasing from 75 ft. At Bohio the channel swings east and is 1,000 ft. wide for 2 miles to Buena Vista; it then swings west and remaining 1,000 ft. wide continues 2½ miles to a point opposite Frijoles. At Frijoles there is another turn to the west and a 1,000 ft. channel to Tabernilla, 2½ miles. At Tabernilla the channel turns east and narrows to a width of 800 ft. for 3 miles to near San Pablo. At San Pablo there is a turn to the west and a 1-mile stretch of 800 ft. channel, then another turn to the right and a 500 ft. channel to Gamboa, 1 mile. This alignment as described seems rather tortuous but the curves are very flat and as will be seen from Fig. 3 the line is more direct than the description makes it appear.

Near Gamboa is Bas Obispo and here the canal is formed by the Culebra cut through the continental divide. At Bas Obispo the channel is reduced in width to 300 ft. and continues at this width for 8.1 miles. As the channel enters the cutting beyond Bas Obispo the height of the banks increases to summits at Contractors Hill of +410 ft. elevation and at Gold Hill of +554 ft. elevation. These two summits are, however, just off the line of the canal; the greatest elevation directly over the canal line is +312 ft. or, since the bottom of the canal is at +40 ft. elevation, the actual depth of cut is 272 ft. The Gatun Lake level of +85 ft. extends through Culebra Cut to Pedro Miguel. The depth of water in the cut is 45 ft. at normal lake level. The canal alignment from Bas Obispo to Pedro Miguel consists of eight reaches, four to the west and four to the east, connected by easy curves.

At Pedro Miguel is the first of the locks dropping the canal from Gatun Lake level to sea level at the Pacific. This lock is a single twin structure with 1,200-ft. dividing wall extensions as at Gatun locks and with the same dimensions of lock chamber. Pedro Miguel lock drops the canal to elevation +55 ft. of Miraflores Lake formed by a dam at Miraflores. This lake is small and the channel through it to Miraflores lock is 500 ft. wide and 45 ft. deep and is 1½ miles long.

At Miraflores two twin locks, with chambers and extended dividing walls of the same dimensions as those at Gatun and Pedro Miguel, drop the level from +55 ft. of Miraflores lake to +10 ft. at high tide or —10 ft. at low tide, which is Pacific sea level. The sea level channel from Miraflores to deep water in the Gulf of Panama is 500 ft. wide and its bottom is at elevation +45 ft. or the depth is 55 ft. at high tide and 35 ft. at low tide. The length of channel from Miraflores to deep water is 7½ miles.

In summary, the canal is 50.4 miles long from deep water in Limon Bay to deep water in the Gulf of Panama. Its length from shore line to shore line is 41.5 miles. There are 14.5 miles at sea level; about 23.5 miles in Gatun Lake; 1.6 miles in Lake Miraflores, some 3 miles in locks, including approach walls, and 8 miles in Culebra Cut. The total curvature is 600° 51' divided between 22 curves.

The alignment of the canal is shown by Fig. 3, and Fig. 7 is a profile giving all essential elevations, distances and other general data. The main structural features of the canal proper which call for particular description are the sea level stretches, the three sets of locks, the Gatun dam and spillway, and the Culebra cut. Besides the canal structures proper there are the harbor works in Limon Bay and the Gulf of Panama.

Lakes Gatun and Miraflores Dams and Spillways

Gatun Lake.

Gatun Lake is the unique feature of the final plan of the Panama Canal, compared with previous plans for the canal. It contributes practically without excavation 23½ miles of navigable waterway. It simplifies the problem of controlling the floods of the Chagres River almost to the point of eliminating that problem. It provides a storage of water for lockage and other purposes which is far more ample than could be provided in any other of the ways suggested.

The first suggestion to dam the Chagres at Gatun is credited by Mr. Joseph Bucklin Bishop, Secretary Isthmian Canal Commission, to Godin de Lépinay, one of the engineer members of the De Lesseps International Congress of 1879. In 1880, in a discussion before the American Society of Civil Engineers, Mr. Ashbel Welch also suggested the Gatun site. Both the Gatun and the Pacific dams were again suggested in 1904 in a paper before the same society by Mr. C. D. Ward. The De Lesseps lock canal plan and its successor, the plan of the New (French) Panama Canal Co., called for dams at Bohio and Bas Obispo. The first Isthmian Canal Commission reporting in 1901, located its dam also at Bohio. This was the plan, tentatively, under which construction was begun by the second Isthmian Canal Commission. Studies were made, nevertheless, of the foundations for a dam at Gatun, by Chief Engineer John F. Wallace, who, however, becoming favorable to a sea level canal, passed the Gatun site over in favor of a dam at Gamboa, near Bohio, which would coincide with a sea level plan. When Mr. John F. Stevens succeeded Mr. Wallace he took up the plan of a dam at Gatun with more favor and had more careful investigations made of its site. He became from his studies an advocate of the Gatun site and urged it upon the International Board of Consulting Engineers, with the result, as has been stated, that the minority report of this board made it an essential feature of its plan for a lock canal.

The feasibility of a great artificial lake formed by a dam at Gatun depended upon four main facts: (1) whether a stable dam was possible of construction at the Gatun site; (2) whether such a dam could be arranged so as safely to waste excess flow of the Chagres in flood; (3) whether such a reservoir could be filled initially, and (4) whether when once filled it could be kept full enough to meet the drafts upon its storage. The first fact and second fact were determined by the engineering studies outlined in the succeeding sections on the Gatun Dam and the Gatun Spillway. The third and fourth will be considered here.

The possibility of being able to fill Lake Gatun initially was from the first a question of time required and not a question of the amount of water available. A study of the flow of the Chagres at Gatun demonstrated that the lowest yearly flow of record would be sufficient to fill the lake in two years. With the beginning of American control careful studies of river flow were inaugurated and in 1911-12 the driest wet season of record was recorded. The average flow for this year was 6,556 sec. ft. This flow without losses will fill Lake Gatun in 400 days.

The ability to keep Gatun Lake once filled, full enough to meet all demands upon its waters depends of course upon the volume of water drafted from the reservoir. These drafts are: (1) Losses due to evaporation; (2) losses due to seepage; (3) leakage through lock and spillway gates; (4) consumption for locking ships to and from summit level; (6) consumption for power, water supply, etc.

Gatun Lake at elevation +80 ft. has an area of 153 square miles, at elevation +85 an area of 163 square miles, and at elevation +90 ft. an area of 173 square miles. Evaporation studies were made and all records of the French engineers of previous years were examined. From the evidence collected it was figured that the probable rate of evaporation would be about ¼-in. per 24 hours, which is a loss of 930 sec. ft. The aggregate seepage loss based on borings and other careful surveys of the lake bed and surrounding highlands was estimated at 85 sec. ft. Leakage through lock and spillway gates was estimated at 275 sec. ft. A similar figure, 275 sec. ft., was determined for the consumption of water for power generating purposes. Lockage is calculated to consume 2,618 sec. ft., based upon the assumption of one lockage per hour in each direction. The total of all these computed drafts on Lake Gatun is 4,183 sec. ft.

The driest consecutive four months in the available records of 19 years to 1908 give a flow of 1,190 sec. ft. In four consecutive months of 1912 the rate was 900 sec. ft. The deficiency between such minimum dry season inflow and the calculated consumption is provided for by storage by filling Lake Gatun in the wet season to +87 ft. elevation. From this level it will be drawn down during the dry season as far as the consumption may demand to a minimum elevation of +80 ft. At +80 ft. elevation there will be a depth of 40 ft. of water in the channel at Culebra Cut.

Gatun Dam.

The minority members of the Board of Consulting Engineers who recommended the lock canal plan, with a dam at Gatun, also presented general plans for the dam. Their belief in the possibility of the dam and in a measure their plans were based on borings of the site by the second Isthmian Canal Commission. Real investigation of the dam site and of the designing and construction problems involved did not begin, however, until Dec. 1, 1907, some time after the canal plan of the minority report had been adopted. These investigations were prosecuted under the direction of Mr. C. M. Saville, assistant engineer, who reported the results Aug. 29, 1908. The report contained some 70 pages of text and 112 plates and diagrams and was a most exhaustive study, and only a very general summary of the findings and conclusions is here possible.

The first work undertaken was to locate a sufficient quantity of material suitable for a dam which was to be built by the sluicing process. Material which appeared suitable was located down the Gatuncillo valley. An additional dredging site was also located in the valley just north of the dam. Previous investigations had indicated a quantity of material in the Chagres valley south of the dam. All these areas were within 1½ miles of the dam and in locations suitable for hydraulic dredging. The total quantity available was estimated as not less than 20,000,000 cu. yds. based on excavation not lower than 50 ft. below sea level. If it should appear that sufficient material was not available in these localities it was asserted that an additional amount could be obtained from excavation in Limon Bay.

The next work undertaken was the investigation of foundations for the dam and spillway. The main questions sought to be answered by these studies were: The ability of the ground to support the structures intended to be built; the likelihood of permeable connection through the alluvial deposits of the gorge across which the dam was to stretch, between the swamps to the south and the flats to the north of the dam; the possibility, with the materials available, of making water tight embankment by hydraulic filling. The investigation was conducted by means of test pits and well drill, diamond drill and wash borings and by building and testing model dams.

A very brief study served to disclose that the outcropping and underlying rock of the gorge was amply strong to support any weight of structures planned. In respect to permeability it was found that some of the underlying and outcropping rocks were of porous texture but that the principal water bearing medium was likely to be the joints and seams which cut through the various strata. It was noted that where this possibility existed cut-off wall construction was easily possible.

The alluvial deposits in the gorges were found to be composed almost wholly of a very fine sand with a large proportion of clay from the surface down to a maximum depth of 50 ft. Under this stratum to a possible depth of 100 ft. was a deposit of blue clay containing little sand and some shells. Underneath this clay for some 20 ft. to bed rock was a stratum of small boulders and angular gravel thoroughly consolidated and cemented together with finely divided clay and silt. This breccia resembled the hard pan or "glacial till" of northern countries. It was waterproof and non-water bearing but when exposed to washing action it broke down and the clay being washed out left a permeable coarse gravel. No continuous layer of loose sand or gravel was disclosed. No deposit of any sort was found extensive enough and sufficiently permeable to endanger the proposed dam.

The experiments with model dams made by hydraulic fill of the materials proposed to be used for the actual dam showed so efficient packing and consolidation of materials by water deposition that seepage was practically eliminated.

Summarizing, the results of the investigations indicated: (1) That suitable material was available near at hand for the construction of the Gatun Dam by the hydraulic process; (2) that the foundations were suitable for such a structure as that proposed at Gatun if they were properly treated; (3) that it was practically possible to construct a stable and water tight dam at Gatun of the materials available; (4) that the hydraulic method of construction as proposed for that work, was feasible if proper conditions were observed.

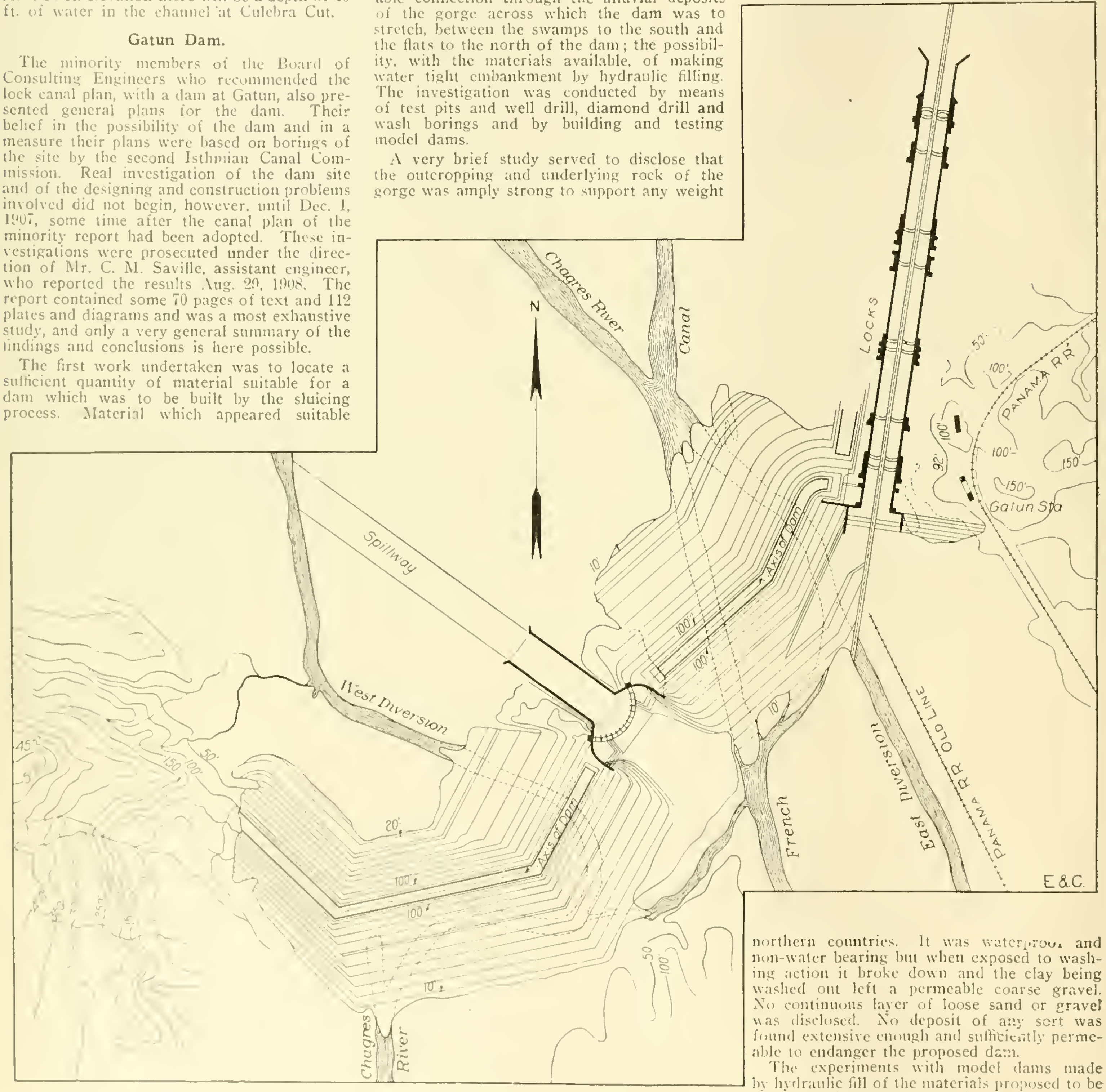


Fig. 8. Map of Gatun Dam and Spillway and Gatun Locks.

Simultaneously with the investigations the commission had proceeded with the design of the Gatun Dam described in the next section. To allay concern felt in the United States as to the safety of the dam as designed, a Board of Engineers was appointed to investigate all features of this work and to report its findings. This board was appointed in January, 1909, and its members were F. P. Stearns, James D. Schuyler, Arthur P. Davis, Isham Randolph, Henry D. Allen, John R. Freeman and Allen Hazen. The board reported Feb. 16, 1909, and the report so far as it related to the dam design and foundations was as follows:

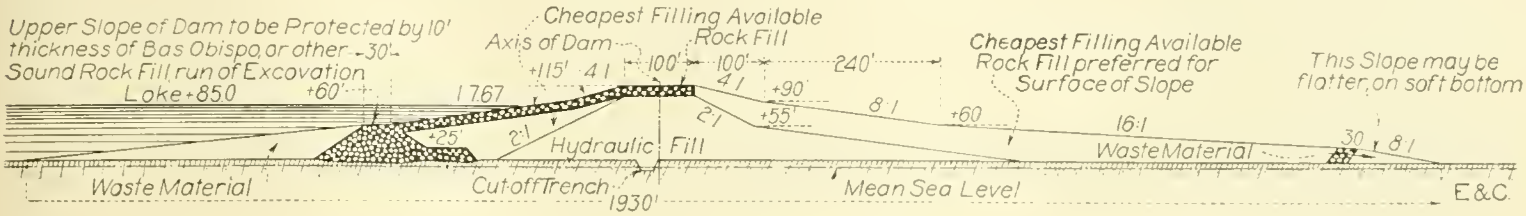

Upper Slope of Dam to be Protected by 10' thickness of Bas Obispo, or other Sound Rock Fill, run of Excavation
30'
Lake +85.0
+60'
1 7.67
+25'
2:1
Cheapest Filling Available
Axis of Dam
Rock Fill
+115'
4 1
100'
100'
240'
4 1
+90
2:1
+55'
8:1
Hydraulic Fill
Cheapest Filling Available Rock Fill preferred for Surface of Slope
+60
16:1
This Slope may be flatter, on soft bottom
30
8:1
Waste Material
Waste Material
Cut-off Trench
1930'
Mean Sea Level
E&C.

Fig. 9. Adopted Cross Section for Gatun Dam.

> We are satisfied, both from the records of the experiments that have been made and from our own personal examination of the materials, as seen in cuts now open, and as disclosed by samples from test borings, that there will be no dangerous or objectionable seepage through the materials under the base of the dam; nor are they so soft as to be liable to be pushed aside by the weight of the proposed dam so as to cause dangerous settlement.
>
> We are also satisfied that the materials available and which it is proposed to use are suitable and can be readily placed to form a tight, stable and permanent dam.
>
> The type of dam now under construction is one which meets with our unanimous approval. It is a combination of rock-fill and hydraulic fill, in which the exterior faces are to be composed largely of rock of all sizes obtained from the canal excavation, dumped and laid on slopes much flatter than are ordinarily found in earth dams, while the interior of the great mass will consist of clayey material obtained by hydraulic dredging from large deposits at a little distance from the dam, and carried by water through pipes to the places where it is to be used. The material as delivered is a mixture of earth and water. The material held in suspension slowly deposits, finally forming a solid, water-tight embankment. The pond necessarily maintained on the top of the dam during construction tests the embankment at all stages of its growth, searches out any weak points and leads to the closure of any voids or cracks.

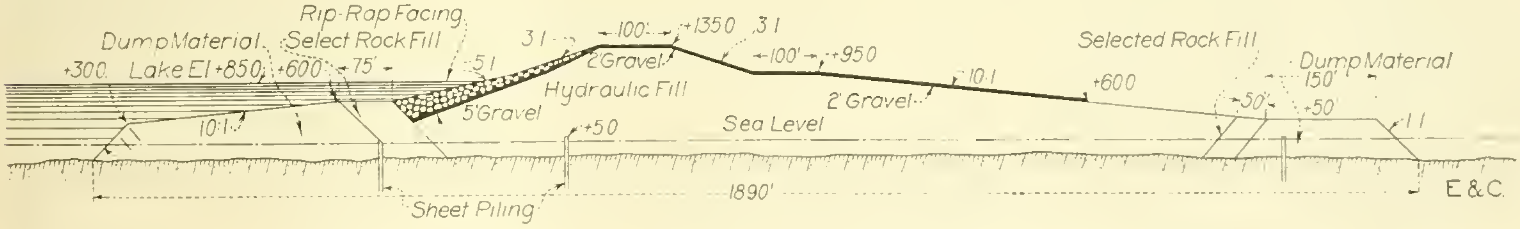


Fig. 10. Preliminary Section of Gatun Dam With Crest Elevation at 135 Feet.

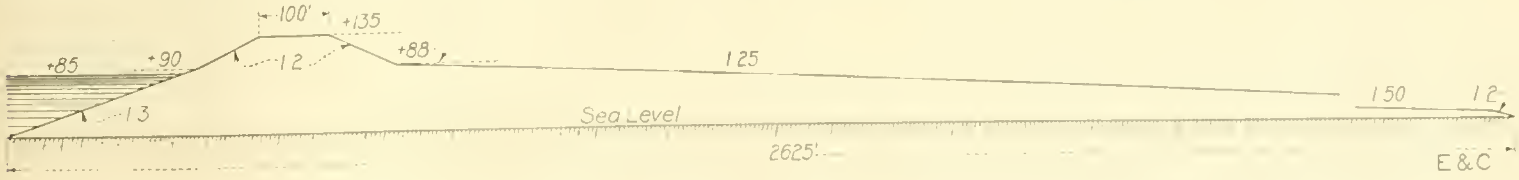


Fig. 11. Section for Gatun Dam Proposed by Minority Report of International Board of Consulting Engineers.

> The most practical question in the construction of the Gatun Dam is the possible slipping and sliding of the materials underneath and in the body of the dam. These materials, speaking broadly, are of a clayey nature, generally impervious to water, but sometimes slipping when subjected to heavy unbalanced pressure or on high, steep slopes when saturated with water. In this respect the materials differ radically from the sandy and gravelly materials which have been frequently used in the construction of other earth dams.
>
> In order to build a dam of these clayey materials that will be stable and permanent, it is necessary that the slopes should be flatter than would be needed to secure the stability of a dam of silicious, sandy or gravelly materials.
>
> The evidence that has been accumulated as to the degrees of slope that are stable with these materials seems to us conclusive. The facts that the materials are slippery does not mean that a dam built from them is necessarily less stable than a dam built of materials that do not slip so easily. It does mean that, in order to secure stability and permanency, the dam must be built with a greater thickness at the bottom. The dam as proposed is more than a third of a mile in horizontal thickness at its base, including the rock-fill portions.
>
> The design upon which the work is now being prosecuted abundantly fulfills the required degree of stability, and goes far beyond the limits of what would be regarded as sufficient and safe in any less important structure.
>
> As a matter of convenience and economy during construction, materials have been piled up on slopes much steeper than those contemplated in the finished work. Generally, the materials so placed have remained in position; but in some cases slips have occurred. The occurrence of these slips is of no serious consequence, either in the practical execution of the work, or in the ultimate stability of the structures. We can readily understand how incorrect deductions may have been drawn from these occurrences, especially by those not fully informed as to the character of the materials, and the ample dimensions and much less steep slopes of the proposed structures in their final form.
>
> We were requested to consider the proper height for the crest of the Gatun dam and after consideration concluded that it could be safely reduced 20 ft. from that originally proposed; namely, to an elevation of 115 ft. above sea level, or 30 ft. above the normal nevel of the water against the dam. We are also of the opinion that the sheet piling recently proposed under the base of the dam may be safely omitted. The narrow cut-off trench now in progress through the upper earth stratum on Gatun island and elsewhere and designed to be refilled with sluiced materials should be continued.
>
> Changes in these respects will facilitate the work of construction, and will reduce somewhat the cost of the proposed work.
>
> A full study of all the data at hand and of the materials, and of the plans that are proposed with the above modifications leaves no doubt in our minds as to the safe, tight and durable character of the Gatun dam.

The various reports of insecurity of the Gatun dam, which had been current in the press of the United States, were practically stopped by this approval of the plans and construction. Construction which had been begun in a tentative way, was now actively pushed and the dam was completed without further serious question of its stability and strength.

EMBANKMENT DESIGN.

Before the report of the board of engineers was made, two plans for a dam had been prepared under the direction of Col. H. F. Hodges, assistant chief engineer. One plan was for a dam with a crest elevation +105 ft. above sea level, and the other was for a dam with a crest elevation +135 ft. above sea level. The recommendation of the board of engineers that the crest elevation be made +115 ft. was adopted. Later the crest elevation was reduced to +105 ft., and a new and final general plan was proposed. The essential features of this plan are shown by Fig 8.

Referring to the plan of the dam site, it is observed that the valley of the Chagres consists in transverse profile of two shallow gorges separated by rising ground or a low ridge. Through the east gorge passed the French canal channel and the East Diversion of the Chagres, and through the west gorge flowed the West Diversion of the Chagres. The plan of dam adopted was earth embankment with rock-fill toes across the gorges and occupying the separating ridge a concrete spillway. The west embankment merged into the hills forming the side of the gorge and the east embankment abutted against the Gatun lock structures. Approximately the dam is 1½ miles long following its crest and nearly a half mile wide at its base. The crest of the structure +105 is 20 ft. above normal water level in Gatun Lake. At water level the embankment is about 400 ft. thick and its crest width is 100 ft. Of the total length only 500 ft. are exposed to the maximum head of water of 55 ft. The adopted cross-section of the dam is shown by Fig. 9, except that as finally constructed the summit elevation was lowered to +105 ft. The section shown by Fig. 10 is that proposed by Col. H. F. Hodges prior to the report of the board of engineers which recommended a lower crest elevation. The sketch, Fig. 11, shows the rough section proposed by the minority report of the International Board of Consulting Engineers. The structural features of the adopted section are indicated on the drawing; they are also described further in the section on the method of embankment construction.

EMBANKMENT CONSTRUCTION.

The general sequence of operations in constructing the Gatun dam was: First, close the French canal and East Diversion of the Chagres, diverting all flow through the West Diversion of the Chagres, and commence filling the east embankment; second, and simultaneously with the first operation, excavate and line the spillway channel to permit the flow of the Chagres to be diverted through the spillway and thus the West Diversion to be closed: third, close the West Diversion diverting the flow of the Chagres through the spillway and commence filling the west embankment; fourth, continue on embankments and on the spillway dam, the filling and concreting required to complete them. This sechedule was followed in general, although some filling which did not require closing the West Diversion was done before the spillway was sufficiently completed to take the flow of the Chagres.

The general method of embankment construction was (1) Extend trestles across the east and west gorges and from them dump stone brought in by train to form the parallel rock fills indicated by the section. Fig. 9; (2) when these parallel rock fills extended across the gorge to pump earth fill between the rock embankments by hydraulic dredging; (3) continue the rock filling and the hydraulic filling upward until the embankment was complete.

In conformity with these general schedules work was begun actively about the middle of 1908 making rock fill and clearing the east gorge for hydraulic fill. Previous to this date a trestle had been driven along the south enrockment from the locks to the West Diversion channel and some 462,297 cu. yds. of rock fill had been deposited from the trestle. During the year to June 30, 1909, trestles were built along the north enrockment of the east gorge and along both enrockments of the west gorge and rock fill carried on from all four trestles. This rock fill was limited in the west gorge because of the necessity of keeping the West Diversion channel open, but in the east gorge it was completed so that hydraulic filling between enrockments could be begun. Altogether during the year ending June 30, 1909. 2,501,372 cu. yds. were placed in the dam, of which 720,017 cu. yds. were hydraulic fill. Adding the 462,297 cu. yds. of enrockment previously completed, there had been placed in the dam at the date last named, 2,963,669 cu. yds.

The method of clearing the east gorge for hydraulic filling was as follows: Stream beds were dredged out to satisfactory sandy clay in the Chagres River, and to original hard blue clay in the French canal. Extreme care was taken with the banks of the old Chagres, which were covered with rip-rap all along the railroad and generally with debris from old Gatun village. The rock and debris were removed and the bank graded down by hydraulic jets and by hand, with a flat slope in the original red clay. A ditch about 20 ft. wide by 15 ft. deep was cut across the valley, extending with about half these dimensions up the hillside. There were 17,222 cu. yds. of excavation in this ditch. The low land of the valley was stripped for a width of 510 ft., the excavation having a depth of 1 to 3 ft., and this stripping was extended up the hillsides with reduced width. Including the material excavated from the river bank the stripping amounted to 35,539 cu. yds.

Hydraulic filling between enrockments in the east gorge was first confined to filling up the old French canal excavation. On April 1, 1909, one dredge began pumping into the Chagres River bed, its work being supplemented about May 15 by that of a second dredge and on June 20 by that of a third dredge. The fill gradually raised, first filling the old river bed and then spreading over the bottom land of the east gorge. The discharge from the dredges of course contained a large percentage of water which had to be drained off as the solid material settled into the fill.

In filling the French canal the surplus water from the hydraulic fill was pumped off; for the balance of the fill the drainage was by gravity through two sets of three 20-in. drain pipes. The discharge pipes from the dredges had of course to extend across the enrockments in order to discharge between them and to prevent the pipes from interfering with the construction of the enrockments which was proceeding simultaneously with the hydraulic filling, the pipes were held below the finished grade. The drain pipes had outfalls into open ditches running to adjacent streams and extended back into the hydraulic fill on a slight up-grade. At their inner ends they projected vertically to near the surface of the water, which, with three hydraulic dredges pumping, rarely ran over 1 ft. above their tops. As the hydraulic fill rose the pipes were extended upward by bolting on 3-ft. lengths of flanged piping.

To summarize briefly before proceeding, since the first preliminary work in July, 1907, to June 30, 1909, two years, there had been placed in the Gatun dam 720,047 cu. yds. of dry fill and 2,238,732 cu. yds. of wet fill, or a total of 2,959,779 cu. yds. at a total cost of $965,595 without charge for plant, but including stripping, trestlework, etc., whose cost should not all be charged to one year's filling. The cost per cubic yard on this basis, however, may well be stated, and it was a little over 32 cts. For succeeding years the cost of the work was better kept and is of more significance.

Until January, 1910, dam construction was confined to the east gorge except for some trestle construction and enrockment work. The three dredges pumping into the east fill were connected with relay pumps to compensate for longer pipe lines and higher lifts which had become necessary as the work advanced. On April 25, 1910, the Chagres, which had been flowing through the West Diversion, was turned into the spillway channel. It was now possible to close the West Diversion channel and to begin active construction of the west embankment. The enrockments across the west gorge were advanced and the site cleared for hydraulic fill. In May, pumping into the west embankment began, and for two months to June 30, 1910, the discharge of all dredges was turned into the west fill so as to make it safe from the floods of the approaching wet season. On June 26, 1910, a fourth dredge was added to the dam filling plant. The clearing done for the hydraulic fill was of much the same character as that for the east embankment and previously described. The area cleared was 62 acres and the excavation amounted to 111,685 cu. yds. In addition there were cleared 138.45 acres in the swamp south of the dam and 51.36 acres in the lowlands north of the dam. These two areas were cleared to give land from which the dredges making the fills could pump.

The embankment work accomplished in the twelve months ending June 30, 1910, was: Dry fill, 2,577,234 cu. yds.; wet fill, 2,933,075 cu. yds. This made the total amount of material placed in the dam, 8,473,975 cu. yds. The cost of the filling for the year was 33 cts. per cubic yard based on a record of 2,555,197 cu. yds.; the cost of the 2,933,175 cu. yds. of dry filling for the year was 29 cts. per cubic yard. With the installation of a complete cost keeping system on July 1, 1909, detailed costs were made available for the final six months' work to July 1, 1910, and they are summarized for both dry and wet filling as follows:

Dry Filling.
(1,558,401 cu. yds.)

Items.	Per cu. yd.
Tracks	$0.0485
Trestles	0.0107
Transportation	0.0630
Filling	0.0627
Clearing site	0.0267
Maint. equipment	0.0037
Plant arbitrary	0.0548
Division expenses	0.0118
Administration	0.0280
Total	$0.3099

Hydraulic Filling.
(1,556,745 cu. yds.)

Item.	Per cu. yd.
Dredging	$0.0952
Relay pumps	0.0053
Pipe lines	0.0336
Wood flumes	0.0001
Power	0.0234
Small boats	0.0009
Maint. equipment	0.0587
Clearing	0.0395
Plant arbitrary	0.0615
Division expenses	0.0072
Administration	0.0245
Total	$0.3499

By the middle of 1910 filling was in progress along the full length of the Gatun dam, and the work from then on consisted of the steady raising of the enrockments and of the hydraulic fill between them. As the embankment rose various changes were necessary in the trestles and track system for making the dry fill and, naturally, in the drainage system and in the arrangement of the dredge pipe lines. Material for dry fill was brought chiefly from Culebra Cut, but also from the Gatun lock and spillway excavations and other nearby sources. The hydraulic fill was obtained from areas north and south of the dam. Three dredges were worked throughout the year, a fourth worked four months, and a fifth for two months. All dredges operated with relay pumps on their discharge pipes. The amounts of fill placed in the dam during the year ending July 1, 1911, were:

Dry fill, car yardage	2,726,094
Wet fill, borrow pit measurement	5,731,096
Totat	8,457,990

This yardage does not mean net yardage of increase in the volume of dam embankment. In respect to dry fill, surveys of the dam showed a decrease of 665,908 cu. yds. from the car yardage delivered. This decrease was due to slides of dry fill into wet fill, to subsidence of dry fill under heavy trains passing over it, and to slow general consolidation. The net volume added to the dam by hydraulic filling was shown to be 1,973,026 cu. yds. less than the borrow-pit measurement given above. This decrease in wet fill was due to general consolidation, waste through drainage pipes, filling of voids in dry fill and compacting of fill, under action of water and pressure, into smaller compass than obtained at the borrow pits. These facts are to be noted in studying the following costs for the year to July 1, 1911; the yardage on which the costs are based are by car and borrow pit measurements.

Dry Filt.
(263,908 cu. yds.)

Items.	Per cu. yd.
Excavation	$0.0096
Clearing site	0.0021
Tracks	0.0530
Trestle	0.0107
Transportation	0.0898
Filling	0.0349
Maint. equipment	0.0316
Plant arbitrary	0.0852
Division expense	0.0144
Administration	0.0286
Total	0.3599

Hydraulic Filling.
(4,256,393 cu. yds.)

Item.	Per cu. yd.
Clearing site	$0.0024
Clearing for dredges	0.0105
Dredging	0.0716
Relay pumps	0.0067
Pipe lines	0.0236
Wood flumes	0.0002
Power	0.0266
Small boats	0.0016
Maint. equipment	0.0270
Plant arbitrary	0.0491
Division expense	0.0066
Administration	0.0182
Total	$0.2441

On July 1, 1912, the condition of the earth dam structure was as follows: East of the spillway for 1,000 ft. the crest was at elevation 103.35 ft., and for the remainder of the east wing the dry fill was at elevation 96 ft., and the wet fill at elevation 85 ft. On the west wing the dry fill was at elevation 98 ft. and the wet fill at elevation 87 ft. There remained to complete the dam 1,652,832 cu. yds. net of fill.

The filling was completed and the embankment portion of the dam was virtually finished during the year ending June 30, 1913. Dry fill was secured from the borrow pit which had been opened during the previous fiscal year, beyond the west end of the dam, and the clay used to top off the hydraulic fill from borrow pits north of the dam and in the vicinity of the locks. From two to six steam shovels were engaged in procuring this material and removed 2,159,159 cu. yds. borrow-pit measurement, of which 922,877 cu. yds. were designated as rock. The hydraulic fill was supplied by three pipe-line suction dredges operated in borrow pits upwards of 1½ miles distant, the maximum lift being 100 ft. The total amount handled, borrow-pit measurement, was 493,115 cu. yds. The hydraulic fill was stopped in September, 1912. No complete survey was made during the fiscal year, but partial cross-sections were run monthly until February, 1913, from which the material in place was calculated, and for the following months, estimates were based on the borrow-pit measurement. Under these circumstances it is estimated that the dry fill deposited aggregated 1,711,367 cu. yds. or a loss of 21 per cent of the borrow pit measurement, due to consolidation. Of the hydraulic fill, it was estimated that there was a loss of 321,141 cu. yds. or 65.8 per cent due to waste through drain pipes, leakage, and consolidation. The total consolidated fill for the year aggregated 1,967,811 cu. yds. The cost of the dry fill in place amounted to $0.3755 per cubic yard, and of the hydraulic fill, $0.2654 per cubic yard.

On the morning of Aug. 20, a bulging and sliding movement began along the north slope of the dam near the west end. At 7 o'clock in the morning of this date the track which was at the top of the dry fill, elevation 101, settled vertically about 3 ft. over a length of about 200 ft., while the track at elevation 63, down the slope, showed slight indications of movement. In the afternoon of the same day a lateral displacement of 8.8 ft. had occurred on the 85 ft. contour and 5.1 ft. on the 50 ft. contour; there was no movement on the 30 ft. contour. On the morning of Aug. 30, a further settlement and bulging had occurred, giving an additional lateral movement of 3.4 ft. on the 85 ft. contour, 3.2 ft. on the 50 ft. contour, while the toe of the slope still remained undisturbed. By the morning of Aug. 31, a still greater displacement had occurred, adding 8.6 ft. to the lateral movement on the 85 ft. contour, 9 ft. on the 50 ft. contour, while the foot of the slope was pushed out laterally 6.2 ft. on the 30 ft. contour. The movement continued through Sept. 1, and ceased Sept. 2. The maximum vertical settlement of the track at elevation 101 was 20 ft., extending for a distance of 750 ft. along the crest, affecting not only the hydraulic fill but the north slope of the dry fill to the north of it. The total lateral displacement along the 85 ft. contour was 33.9 ft., on the 50 ft. level 31.2 ft., and on the 30 ft. contour 18.1 ft. The south slope of the dam remained undisturbed. The slope of the portion of the dam which moved was originally fixed at 1 on 5, and this was authorized "because the rock is near the surface and fairly horizontal. The only thing to be provided against in this slope is the slipping of the dam material on itself." Because of the character of the material along this portion underlying the structure there could be no question that the movement was within the dam itself. A line of wash borings with drive samples was made. These borings clearly indicated that the relative proportions of hydraulic fill and dry fill which would bring about the desired section of hydraulic fill—wedge shape, with the point down—had not been secured, but that on the contrary the hydraulic fill in section was the opposite of this. Furthermore, it was evident that provision had not been made against the slipping of the dam material itself. As in the case of the movement on the north face of the east portion of the dam, the toe was heavily reinforced and the slope flattened to an average of about 1 on 7.67.

To prevent injury to the dam that might result from wash of the south slope by waves in the lake caused by strong south winds, which prevail at times, it was necessary to pave such portion of this slope as is liable to such action. Concrete paving was at first considered, but because of settlement, due to consolidation and the irregularity of the surface, it was decided to use rip-rap laid on broken stone. It was estimated that waves 5 ft. in height at times exist, so that the paving was extended over that portion of the slope lying between elevations +71 and +92. A layer of crushed stone, supplied by Ancon quarry, was laid over the dam within these limits to a thickness of about 4 ins. Over this a sufficient thickness of rip-rap was placed to protect the broken stone from the action of the waves. The rip-rap was at first procured from a quarry in the vicinity of Quebrancha and was estimated to cost $1 per cubic yard in place on the dam. As the cost however, after a trial of two months, amounted to $2.10 per cubic yard in place, and as the stone was mixed with a considerable amount of clay, this quarry was abandoned and the stone was furnished from the excavation for

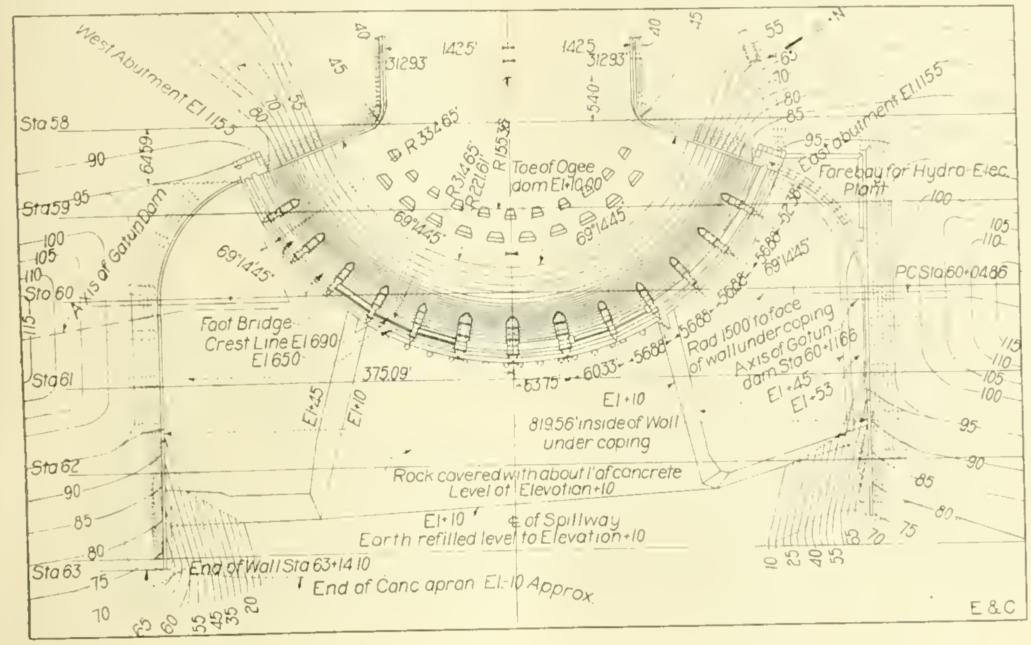

West Abutment El 115.5
Sta 58
Sta 59
Sta 60
Sta 61
Sta 62
Sta 63
Axis of Gatun Dam
Toe of Ogee dam El+10.80
East Abutment El 115.5
Forebay for Hydro-Elec. Plant
PC Sta 60+04.86
Foot Bridge
Crest Line El 69.0
El 65.0
375.09'
El+10
819.56' inside of Wall under coping
Rod 1500 to face of wall under coping
Axis of Gatun dam Sta 60+11.66
Rock covered with about 1' of concrete Level of Elevation+10
El+10
of Spillway
Earth refilled level to Elevation+10
End of Wall Sta 63+14.10
End of Conc apron El-10 Approx.
E&C

Fig. 12. Plan of Overflow Wier of the Gatun Spillway.

Dry Fill.
(2,602,073 cu. yds.)

Items.	Per cu. yd.
Clearing site	$0.0007
Excavation	0.0724
Tracks	0.0852
Trestles	0.0038
Transportation	0.1209
Filling	0.1010
Maint. equipment	0.0157
Plant arbitrary	0.0690
Division expenses	0.0187
Administration	0.0359
Total	$0.5533

Filling operations for the succeeding year to July 1, 1912, followed practically the routine of the previous year. A total of 9,048,896 cu. yds. of material, borrow pit measurement, were placed and this consolidated in the dam to 5,525,945 cu. yds., which is a shrinkage of 35.9 per cent. The wet fill and dry fill were more nearly equal in volume than in previous years, being, respectively, 2,982,859 cu. yds. and 2,543,056 cu. yds. The cost of filling during the year was as follows:

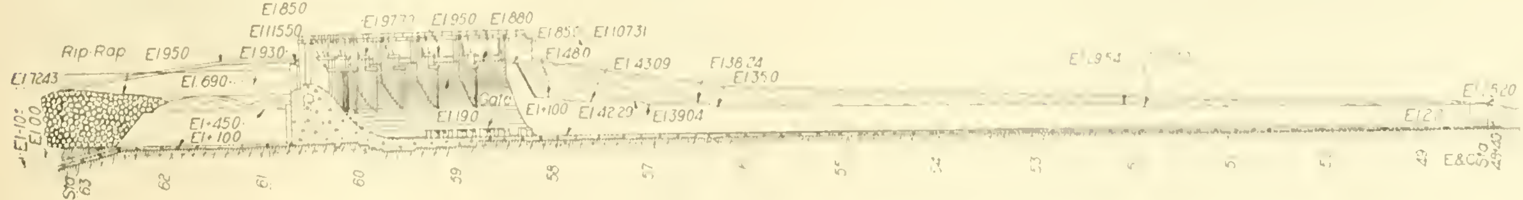


Fig. 13. Longitudinal Section of the Gatun Spillway.

Hydraulic Fill.
(2,604,632 cu. yds.)

Clearing site	$0.0007
Clearing for dredge	0.0117
Dredging	0.1244
Relay pumps	0.0132
Pipe lines	0.0257
Wood flumes	0.0028
Power	0.0762
Small boats	0.0028
Maint. equipment	0.0169
Plant arbitrary	0.0725
Division expenses	0.0075
Administration	0.0201
Total	$0.4045

the dry dock at Sosa Hill. An area of 115,740 sq. yds. was covered with crushed stone, of which 15,740 cu. yds. were used, completing this portion of the work in April at a cost of $2.4536 per cubic yard in place. The riprap stone placed at the close of the fiscal year aggregated 68,730 cu. yds., covering an area of 102,030 sq. yds. at a cost in place of $0.6574 per cubic yard.

A summary of the embankment fill measured in place in the dam gives the following figures:

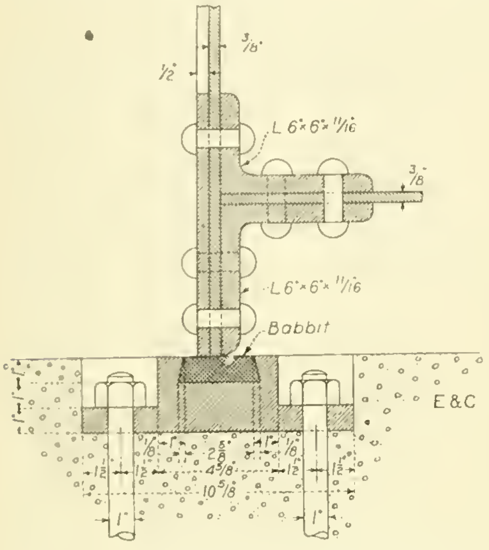


Fig. 14. Detail of Bottom Seal Stoney Gates for Gatun Spillway.

Year.	Dry.	Wet.	Total.
Prior to July 1, 1908	462,297	....	462,297
1908-09	1,781,325	720,047	2,501,372
1909-10	2,577,234	2,933,075	5,510,309
1910-11	2,060,186	3,758,870	5,819,056
1911-12	2,982,859	2,543,086	5,525,945
1912-13	....	....	1,967,841
Total	....	....	21,786,820

Gatun Spillway.

During the rainy season the influx of water into Lake Gatun exceeds the amount consumed as described in the section on Lake Gatun. To provide relief for this excess flow a spillway was necessary, and, as already stated, suitable location for this spillway was found in the rock ridge or hill which divided the Chagres valley at Gatun into two gorges. The plan of Gatun dam, Fig. 8, shows this location of the spillway, and also indicates its main structural parts, namely, a concrete dam, with means for overflow, and a channel to lead the water to the River Chagres below the dam.

DESIGN AND STRUCTURAL FEATURES.

The maximum known discharge of the Chagres River for a period of 33 hours is 137,000 sec. ft.; its maximum momentary discharge is 182,000 sec. ft. It was estimated that a spillway to handle such floods, with uncontrolled overfall, would need to be 2,000 ft. long and the lake would rise 5 ft. in extreme flood. Because of the cost of so long a spillway dam and because of other objections, it was from the first decided to employ a weir with crest gates, or controlled overfall. The general design worked out is shown by Figs. 12 and 13. As will be seen crest length was gained by throwing the trace of the dam into a circular arc, and controlled overflow was secured by means of gates along the crest. The structural features of this design and its efficiency under the conditions may be explained as follows:

Because of the circular form of crest the discharge is directed toward the center of the crest arc where the energy of the converging stream partly neutralizes itself. To complete the neutralization, two rows of baffle piers are placed on arcs of circles concentric with the crest arc, the upper row being about 120 ft. downstream from the crest. These baffle piers were built of concrete faced on the upstream side with cast iron plates, and they project about 10 ft. above the surface of the apron. The dam proper has an ogee section composed of a parabola,, a short tangent and a circular arc leading to the flat apron below the dam. The curve planned is such that when the stream of water flowing over the crest is 6 ft. or more in thickness, the nappe will adhere to the downstream face of the dam. The crest of the dam is divided into 14 openings, each 45 ft. wide, by 13 piers and 2 abutments. In these openings are placed Stoney gates constructed and operated as described in a succeeding paragraph.

The sills of the Stoney gates were placed at elevation +69 or 16 ft. below the normal level of the lake, which was assumed at elevation +85. It is planned not to allow the level of the lake at Gatun to exceed elevation +87, and with the lake at this level one opening will discharge 11,000 sec. ft. and all 14 openings will discharge 154,000 sec. ft. This it will be seen is an amount in excess of the 137,000 sec. ft. maximum known discharge of the Chagres for 33 hours. The gates of the spillway when fully raised have their bottoms at +92 ft. elevation; allowing for the slope of the water in passing over the weir, the clearance of the gates with lake level at +87 ft. is estimated to be 7 ft. This clearance was considered ample to take care of drift.

The Stoney gates are cellular steel structures 19 ft. high, 46 ft. 3½ ins. wide and about 55 ins. thick. Their framework consists of four horizontal plate girders trapezoidal in outline fastened at their ends to two vertical end girders. The framework carries an upstream and downstream sheathing of ⅜-in. steel plate. The notable details of the gate construction are the bottom seal, Fig. 14, and the end mechanism, Fig. 15. The machinery for operating the gates is located in the tunnel shown in section by Fig. 13, and may be described briefly as follows: Two chains pass vertically, one from each end of the gate, over sheaves mounted immediately beneath the footway at the masonry piers between which the gate rises and falls. After passing over the sheave the descending branch of the chain is made fast to a screw, from the lower end of which hangs a counterweight. Between the end of the chain and the counterweight the screw passes through a revolving nut which forms the hub of a worm wheel; the worm wheel is equipped with double thrust bearings and is capable of taking a vertical thrust in either direction. The two worm wheels, one for each end chain, are actuated, one by a right and one by a left worm, the worms forming the ends of the shaft of a motor placed in the tunnel, and in the plane of the middle of the gate which it operates. As the motor turns, both the worm wheels revolve, causing the corresponding vertical screws to travel both in the same direction and at exactly the same rate, thus raising or lowering the gate. The screws themselves can not turn, being held by the counterweights, which travel in guides. The effect of the counterweights is, of course, to diminish the power necessary to move the gate. The gate itself is 46x19 ft. and weighs approximately 43 tons. As the head under which the gates operate may reach 18 ft., trains of live rollers are provided to diminish the friction against the side castings. On these roller trains the gate rolls in its up-and-down travel, the trains moving at the same time in the same direction as the gate and with half its speed. The roller trains are suspended by a chain passing around a sheave, the dead end of the chain being fastened to the pier near its top and the live end to the gate itself. When the gate is raised and the water rushes under its lower edge, it is thought desirable to have means of hoisting the roller trains out of the current, as otherwise they would remain projecting for about one-half their length below the lower edge of the gate, and might be injured by drift or floating obstructions. For this purpose a device for moving the roller train at a more rapid rate near the end of the travel of the gate has been designed. It consists of a pair

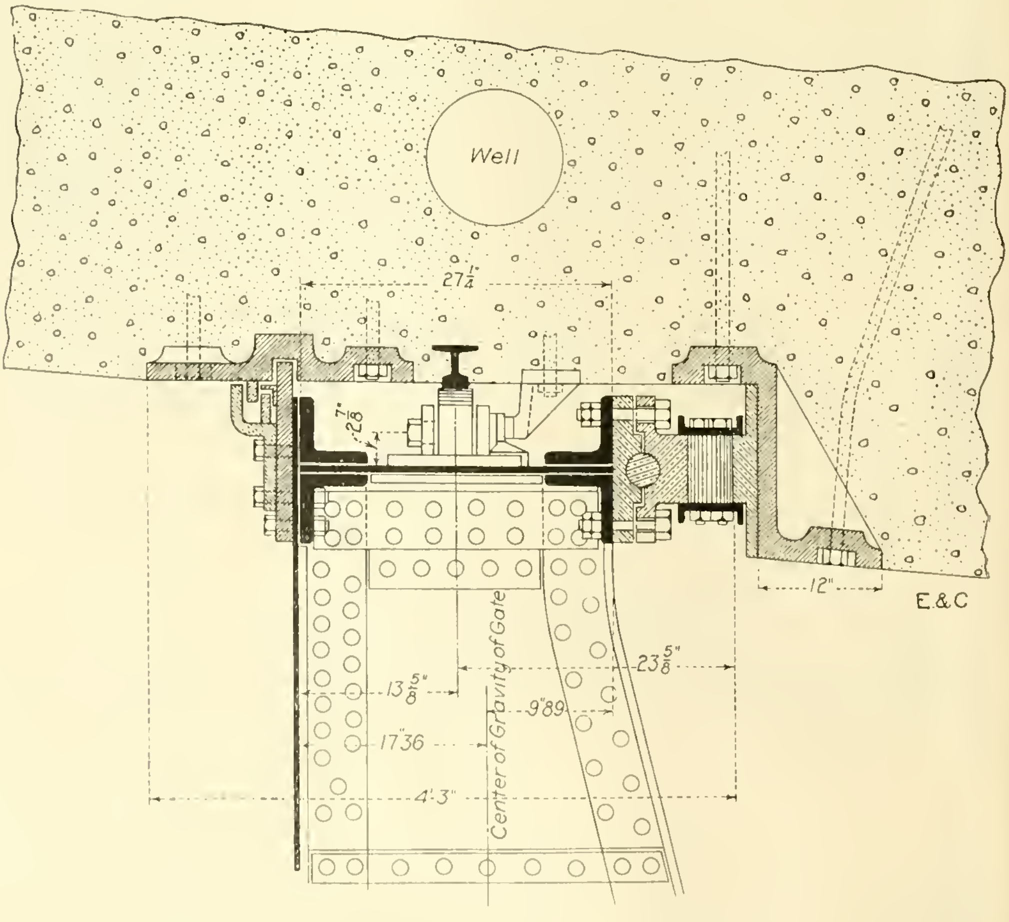


Fig. 15. Detail of Side Bearing and Water Seal of Stoney Gates for Gatun Spillway.

of sheaves placed near the top of the gate, around which the chain of the roller train passes, both sheaves remaining fixed until the bottom of the gate is above water and frictional risistance eliminated. The gate is then guided upstream by two pairs of guide wheels so that the roller train is freed from contact with the downstream face of the gate. At this point in the upward travel the lower sheave engages and is held by a bracket in the masonry pier while the upper one continues to rise with the gate. The chain between the sheaves is therefore drawn out and the speed of motion of the roller train correspondingly increased, so that at the end of the travel the bottom of the gate and the bottom of the roller train will be at the same elevation and the train will be protected from accident.

The preceding description has referred largely to the spillway weir proper. The water, after passing over the weir, enters a fan-shaped chamber which converges the flow into the spillway channel. This is a concrete lined channel 285 ft. wide and 1,200 ft. long, excavated through rock to the Chagres River.

CONSTRUCTION METHODS AND COSTS.

The general plan of dam construction called for the diversion of the Chagres through the spillway at the earliest possible moment. Excavation of the spillway channel was therefore begun in 1907, and in the twelve months to July 1, 1908, a total of 938,901 cu. yds. of rock were excavated. The excavation of the succeeding year was 359,821 cu. yds., and that for 1909-10 was 127,610 cu. yds. The total excavation of the three years was 1,426,332 cu. yds., or about 94 per cent of the total spillway excavation. The bulk of the excavation was done by drilling and blasting. The broken rock was excavated by steam shovels and loaded into cars which were hauled by locomotive and their loads dumped into the enrockments of the Gatun dam. The 938,901 cu. yds. excavated to July 1, 1908, cost about 40.7 cts. per cubic yard to excavate. The cost of succeeding years' excavation was higher because it consisted largely of excavating by hand and smoothing the rock bottom of the channel to receive the concrete pavement, and because it was interrupted by overflows. Hand excavation cost from 65 cts. to $1.90 per cubic yard.

The concrete paving was begun March 17, 1909, and by April 25, 1910, the floor, side walls, curtain walls and spillway weir foundations were sufficiently completed to permit the diversion of the Chagres through the spillway channel. The total amount of concrete laid to July 1, 1909, was 30,464 cu. yds., and its cost in place was $6.07 per cubic yard. Owing to the necessity of placing the mixing plant at a point of economical delivery of materials by water, the mixed concrete was hauled an average of 4,520 ft. from mixer to place of deposit. A double track service railway was laid, one track for loaded trains and one track for empty trains; two car trains each hauling two 2 cu. yd. batches were operated. For the year to July 1, 1910, 53,632 cu. yds. of concrete were placed, making a total of 84,096 cu. yds. costing $8.71 per cubic yard in place.

After turning the Chagres through the spillway, construction work was possible, of course, only above water level, or by protecting the working space from water. The main portion of the concrete work not above water level and not completed prior to turning the Chagres through the channel was in the spillway dam or weir. The plan of building this was as follows: First, before the water was turned through the channel the foundations for the dam were completed to elevation +10 ft. Also, small piers, as shown in Fig 12, were built to above low water level, which was set by the dry season flow of about 3,000 sec. ft. By setting stop logs between these small piers a shield or coffer was formed behind which were built the three middle piers, Fig. 12, each of which contained a culvert. After completion of these culverts the dry weather flow was diverted through them, making it easily possible to lay dry the other parts of the dam foundation and to build the dam upward. When the dam and the gate piers were completed, the gates were set in all the openings except those between the three center piers. These center openings and the culverts in the meantime carried the flow.

The next step was to close the culvert openings; they were closed with stop logs and the interior culvert valves were removed and the culverts were filled with concrete. The next step was to erect the two center crest gates and they were erected one at a time by erecting a timber bulkhead across the opening and bearing on the upstream sides of the piers.

During the two dry seasons, from June 30, 1910, to July 1, 1912, the spillway weir construction was carried on as described above. Also work was prosecuted on the wing walls and other parts as the water permitted. By July 1, 1912, the excavation for the entire spillway was completed and amounted to 1,594,915 cu. yds.; it had cost an average of 70.9 cts. per cubic yard for excavation proper about 1,544,202 cu. yds., and for 44,091 cu. yds. of trimmings and preparing foundations it had cost $2.19 per cubic yard. The weir proper had been completed, including the abutments, ogee, and crest piers to full height of +69 ft. elevation, except for 370 ft. at the center, which was completed to elevation +59 ft. Altogether 202,413 cu. yds. of concrete had been laid at an average cost per cubic yard of $7.86. During the year ending June 30, 1913, 21,719 cu. yds. of concrete were added, making a total of 224,132 cu. yds. at an average cost of $7.48 per cubic yard. On this date the concrete work was some 95 per cent completed.

About one-half of the concrete for the spillway or 118,317 cu. yds. was laid in the two years, July 1, 1910, to July 1, 1912, and the itemized costs for these two years are as follows:

COST OF SPILLWAY MASONRY.

Item.	1910-11.	1911-12.
Cubic yards	59,651	58,048
Concrete	$4.5324	$4.7461
Large rock	0.0377	0.0094
Wood forms	0.5924	0.7360
Placing	0.6170	0.6287
Reinforcement	0.0077	0.0439
Pumps	0.0032	0.0041
Cofferdams	0.0333	0.0311
Maintenance equipment	0.2234	0.2856
Plant, arbitrary	0.5399	0.4200
Dinner expenses	0.1174	0.1084
Administration	0.2829	0.4105
Total	$6.9873	$7.4238

EFFICIENCY OF THE SPILLWAY.

The ability of the Gatun spillway to hold control of any possible flood is now to be considered. With lake elevation at +87 ft. the spillway discharge is 154,000 sec. ft. As stated, the maximum momentary discharge of record of the Chagres is 182,000 sec. ft., and the maximum discharge for a period of 33 hours is 137,500 sec. ft. Since the coping and the gate tops at upper Gatun and at Pedro Miguel are at elevation +92 ft., it would require a rise at Gatun Lake of 5 ft. above level +87 ft. to do material damage. It would take a discharge of the Chagres of 137,500 sec. ft. about 9 hrs. 20 mins. to raise the level of the lake 1 ft. of from +87 ft. to +88 ft., even were there no discharge through the spillway. Were it possible for the lake to reach elevation, +92 ft., when it would first become dangerous, the spillway dam, if fully opened, would discharge 222,000 cu. ft., which is 40,000 sec. ft. more than the maximum momentary flood discharge of record. Finally by opening the sluices in the locks at Pedro Miguel and at Gatun, an additional discharge of 40,000 sec. ft. would be obtained.

Accident to one of the Stoney gate leaves is remotely possible. To permit repairs in such a contingency or to meet ordinary service needs a caisson is provided for closing off any gate opening. This caisson is to be floated to a bearing against two upstream pier faces. The structure is a steel box about 6 ft. wide, 21⅓ ft. high and 46½ ft. long. The bottom of the caisson and its ends are provided with bearing timbers to increase water tightness when the caisson is in use. One caisson is installed at Gatun and another at Miraflores.

MIRAFLORES LAKE DAMS AND SPILLWAY.

Miraflores Lake forms the first level below the summit level on the Pacific side. Like Lake Gatun it is an artificial reservoir. It occupies an irregular shaped basin between the Pedro Miguel lock and the Miraflores locks, and is formed by impounding the Rio Grande River and some small tributaries by dams at the north and south ends. Its water surface is at elevation, 55 ft. or 30 ft. below normal Gatun lake level.

The dams which impound Miraflores Lake consist of an earth fill dam at Pedro Miguel (Fig. 16), and at Miraflores of an earth fill dam and of a concrete spillway dam (Fig. 17). The dam at Pedro Miguel extends from the west lock wall back almost parallel with the channel approaching from the northwest to adjacent highlands into which it merges. It consists of rock fill toes filled between with earth compacted in layers. Its crest elevation is +105 ft. The earth dam at Miraflores extends from the west lock wall to the high grounds of Cocoli Hill. Its crest makes an acute angle with the lock wall and is at elevation +70 ft. The concrete dam at Miraflores is on the opposite side of the locks from the earth dam, and extends from the east upper lock wall across the old channel of the Rio Grande to the high east bank of that stream.

The Miraflores spillway is in all general features, except that its crest is a straight line, similar to the Gatun spillway. It, however, has only eight gate openings; its spillway crest is at elevation +38.67. The construction of this spillway was a safety precaution. A very small waste weir would have been ample to care for the diverted run off from the drainage basin tributary to the lake and the water from the locks at Pedro Miguel, but it was determined to employ a spillway large enough to insure protection in case of accident by which flow should be established from the summit level through the Pedro Miguel lock into Miraflores Lake. The governing condition was therefore imposed that the spillway should be able to pass all the water which would flow into the lake should one of the twin locks at Pedro Miguel be thrown completely open to the current from the summit level without allowing the level of Lake Miraflores to rise sufficiently to overtop the upper gates and lock walls. The spillway was therefore, designed to pass 90,000 sec. ft. Its crest length is 432 ft.

The method of construction employed fo the Pedro Miguel dam was dry filling, th earth being selected material deposited an compacted in layers. There was a total o 696,558 cu. yds. in the dam, and the cost i place was 44.7 cts. per cubic yard. The eart dam at Miraflores was, like the Gatun dam constructed by hydraulic filling between enrockments. There were 625,048 cu. yds. of hydraulic fill, and approximately 1,600,000 cu. yds of concrete, of which 435 cu. yds. were reinforced; plain concrete cost $5.76 per cub yard and reinforced concrete cost $19.60 pe cubic yard.

All construction work on the Gatun Dam and Gatun Locks came within the Atlantic Div sion and was in charge of Col. William L Sibert, division engineer, with Maj. Cheste Harding as principal assistant until he wa transferred Feb. 27, 1913. Other engineers responsible positions under Col. Sibert wer Maj. Edgar Judwin, Maj. J. C. Harvey, Cap G. M. Hoffman, Capt. H. W. Stickle, Capt. W S Ross and Lieut.-Col. William V. Judson.

The construction of the Miraflores Dam and spillway came within the week of th Pacific Division of which Mr. S. B. Williamson was division engineer until Jan. 1, 1913 when he resigned and the work was take over by Col. Goethals and assigned to one the divisions of the chief engineer's office.

Locks and Lock Operating Appliances

Lock Structures.

The Panama canal has three sets of locks located at Gatun, at Pedro Miguel and at Miraflores. The lock unit adopted is a twin lock, with chambers side by side, separated by a common wall, and standard chamber dimensions of 1,000x110 ft; the chambers are deep enough for vessels of 40 ft. draft. The locks at Gatun consist of three units, those at Miraflores consist of two units and those at Pedro Miguel consist of a single unit. The two and three units forming the locks at Miraflores and at Gatun are set end to end, forming a continuous flight. The lock chamber dimensions given are the greatest of any canal locks in the world, and are 100 ft. greater in length and 15 ft. greater in width than were recommended in the minority report of the International Board of Consulting Engineers.

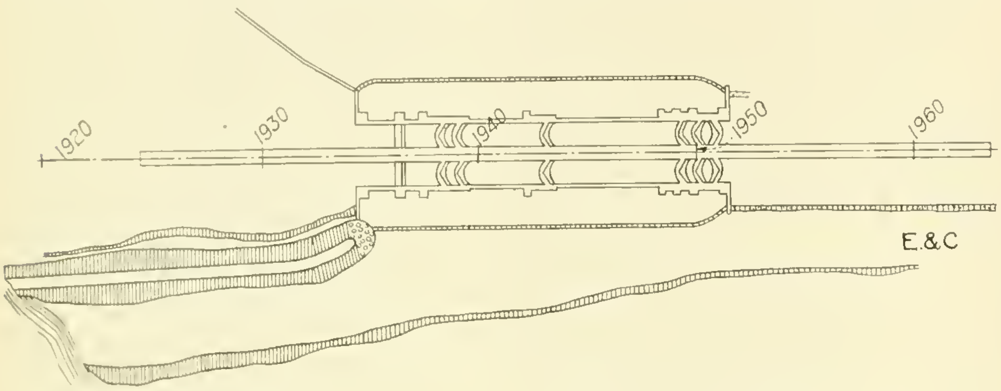


Fig. 16. Diagram Plan of Pedro Miguel Locks and Dam.

GENERAL DESCRIPTION.

The location and arrangement of the three sets of Panama locks are indicated by Fig. 8 and Figs. 16 and 17. The diagram plans, Fig. 18 show the chamber dimensions and the location of gates, fender chains and emergency dams. Since the general structural features of all locks were the same they are shown only for one unit of the Gatun Locks, Fig 19. The locks have concrete walls and floors which are suitably recessed for gates, sills, etc. Where the locks are in flight, the walls and floors are continuous. The center or dividing walls are also in each case extended from 1,000 to 1,200 ft. beyond the lock ends to form guide and temporary mooring walls. All lock masonry is founded on rock and in case of portions of the Gatun Lock floors the masonry is anchored to the foundation work.

To fill and empty the lock chambers culverts running lengthwise through the bases of the walls are provided. There are three such culverts, one in each side wall and one in the middle wall, each having a sectional area of 251 sq. ft., the equivalent of a circle 18 ft. in diameter. Each longitudinal culvert is controlled by a rising stem Stoney gate valve. In the sidewall culverts the intakes at the upper end of the wall are provided with guard valves of this type, and the main culvert is provided with service valves at the upper, middle and lower gates. At the location of the service valves the culvert is divided into two parts by a middle pier, it being considered inadvisable to attempt to control the whole culvert with one valve. The valve at the lower gates is duplicated. The side wall culverts open directly into lateral culverts, eleven in number, each of these laterals being 11 sq. ft. in area. The lateral culverts communicate with the chamber by openings through the floor, there being in each lateral five such openings, each with an area of 12 sq. ft. The middle wall culvert is controlled by duplicate sets of Stoney valves at its upper and lower extremities. It communicates with the lock chambers on either side through lateral culverts, eleven in number, similar to the side wall laterals, except that each one is 33 sq. ft. in area at the throat and is controlled by a cylindrical valve at its junction with the main culvert. This control makes the twin lock chambers independent. The longitudinal culverts are continuous the full length of each lock flight.

The culvert area is such that if all valves be opened at once the lock chambers can be filled or emptied in a very short time. The currents set up above and below the locks and in the lock chambers by fully opening all culverts might cause damage to vessels, however, and it was assumed that the water level in the locks should not change to exceed 3 ft. per minute, and under this assumption the culverts will fill or empty the locks in about 15 minutes. Except when the full 1,000-ft. lock chamber is required only the side culverts will be used.

As seen by the diagram plans, Fig. 18, the locks have duplicate gates, guard gates and intermediate gates, which permit of various subdivisions in length of lock chamber. All gates are mitering steel girder gates as fully described in a succeeding section. Referring to the various lock chamber arrangements possible: The upper lock chambers at Gatun and Miraflores and the lock at Pedro Miguel, as shown by Fig. 18, have two sets of service gates at each end as a guard against accident. For these units, measured from the hollow quoin of the lowest set of gates to the hollow quoin of the set farthest upstream the length of lock chamber is 1,181 ft. The useful length of these locks for a vessel passing downward is assumed to be between the protective chain guarding the lower gates and the hollow quoins of the upper service gates. Measured from this protective chain to the quoins of the lower set of upper gate, the clear distance is 908.5 ft., and, measured to the quoins of the upper set of upper gates, the clear distance is 1,000 ft. In all lockages of vessels of 900 ft. or less in length, both sets of upper and lower gates will be operated simultaneously. In locking downstream a vessel between 900 and 1,000 ft. long, both sets of lower gates will operate together, while the lower set of upper gates will be left open, allowing the stern of the vessel to project into the space between the two sets of upper gates. In locking similarly large vessels upstream, a similar gain may be made by using the space between the two lower gates, the two sets of upper gates being operated together. A vessel thus always has two gates in front of it, separating the levels. A middle gate with its hollow quoin 550 ft. above the lower protective chain, divides the upper lock chambers into two. Similar intermediate gates in the chambers of the lower lock units divide these chambers into two. A single gate separates the middle from the lower lock unit at Gatun. The downstream lock units at Gatun and Miraflores and the single unit at Pedro Miguel have at their lower ends guard gates besides the service gates. There is also above the lower service gate at Pedro Miguel a safety gate.

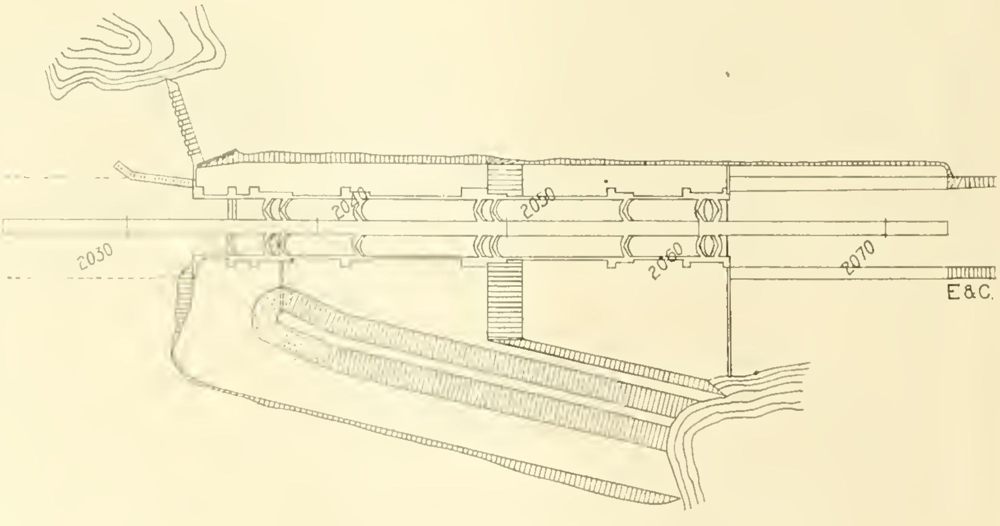


Fig. 17. Diagram Plan of Miraflores Locks, Dam and Spillway.

The lock structures comprise in addition to the chamber structures and gates an emergency dam for each set of locks. This dam is located above the locks, and its purpose is to close the channels leading to the locks whenever such closure is desirable for any purpose. The lock structure also includes operating machinery for gates and valves and for emergency dams and at each lock there are electric locomotives for towing vessels through the locks.

CONSTRUCTION METHODS.

The construction of the canal locks involved four general classes of work: (1) Excavation; (2) sand and stone supply; (3) concrete construction for lock walls and chambers, and (4) gate, valve and machinery erection and installation. Since the method of operation was different for each set of locks, the work at Gatun, Pedro Miguel and Miraflores will be described separately.

EXCAVATION.

Gatun Locks.—The Gatun locks, as shown by Fig. 8, occupy a site on the flank of the eastern hills bordering the Chagres valley at Gatun. The highest elevation of the natural surface on the site was about +30, and the lowest was approximately +5. Borings showed, beginning at the south end of the site, a rising stratum of argillaceous sandstone, whose lower slopes were covered with clay, but which became exposed near to the north end of the middle locks. Under this stratum was a stratum of conglomerate, and a stratum of soft sandstone, and then argillaceous sandstone mixed with tufa. The depth of the uppermost argillaceous sandstone stratum was enough to provide for the lock foundations within itself, except at the extreme upper end of the upper locks whose foundation floor cut into the conglomerate and soft sandstone. The soft sandstone was found by test pit to be slightly water bearing and as an extra precaution the portion of the upper lock floor which came closely over this standstone was planned to be anchored down as shown by Fig. 19. Careful tests were made which determined first that the compressive strength of the rock was ample to withstand the lock loads and second that this rock provided ample holding power for steel rail anchors. The stone withstood compression loads of 500 lbs. per square inch and 5 ft. and 10 ft. rail anchors required pulls to move them of 130,000 lbs. to 200,000 lbs.

The excavation was begun in 1907 and was performed by channeling the sides of the pits, drilling and blasting the rock cores between channel cuts, excavating and loading the spoil by steam shovel and hauling the spoil by train to make enrockment in Gatun Dam. As the final levels were approached hand excavation was employed to prevent shattering of the rock floor and unfitting it to receive the concrete footings and floors. Rock excavation was virtually completed on June 30, 1911, and amounted to the following quantities:

Method.	Cu. yds.	Unit cost.
Steam shovel	4,555,395	$0.6048
Hand	186,425	1.7284

The overburden of clay which covered the sloping rock beyond the south end of the dam site had to be removed to rock for portions of the flare walls and the south approach wall. This was done by dredging, and it cost for some 975,000 cu. yds. about 32 cts. per cubic yard, including the cofferdams required.

Pedro Miguel Locks.—Borings on the site of the Pedro Miguel locks showed firm rock to exist well below the lock bottom. The method of excavation was essentially the same as that employed at Gatun and the spoil was deposited in the Pedro Miguel dam. The total volume of steam shovel excavation was 1,130,236 cu. yds., and it cost 91.14 cts. per cubic yard. The hand excavation preparing foundations amounted to 160,621 cu. yds. and cost $2.73 per cubic yard.

Miraflores Locks.—Borings on the site indicated hard limestone under the upper locks and argillaceous limestone under the lower lock. As at Gatun and Pedro Miguel, excavation methods were channeling, drilling and blasting, steam shovel and hand excavation. Spoil from the site was deposited in the Miraflores dam. The steam shovel excavation in the lock site amounted to 2,222,582 cu. yds., and cost 92.9 cts. per cubic yard. Hand excavation preparing foundations amounted to 366,933 cu. yds. and cost $1.83 per cubic yard.

STONE SUPPLY.

Rock suitable for concrete could not be obtained from the canal excavation except at the expense and trouble of careful sorting. It was decided therefore, to develop special quarries. The considerations to be regarded in selecting these quarries were: That the quantity of stone available should be ample, that its quality should be suitable, and that the location should be such as to permit of the most economical transportation from the crusher plant to where the stone was to be used.

Gatun Locks.—A quarry location at Porto Bello, about 20 miles east of Colon, best satisfied the conditions governing stone supply for the concrete lock and spillway work at Gatun. Here a quarry of ample volume and of good material could be developed close to the shore and its product could be transported in barges to Colon harbor and up the old French Canal to unloading wharves and storage piles adjacent to the Gatun Locks. Some 2,000,000 cu. yds. of stone were needed for the Gatun work and explorations at Porto Bello indicated some 20,000,000 cu. yds. available of good andesite rock. The quarry site was also favorable, being a hillside rising directly from the water on the north shore of Porto Bello Bay.

The general development of the plant was as follows: Stone was to be taken only above elevation + 88, which was made the level of the crusher house floor, as the rock above this level to a vertical face along the + 160-ft. contour would give 4,000,000 cu. yds. The crusher house was 53x70½ ft.; down the slope at elevation 40 ft. was located a 51x57½-ft. engine house and further down at about elevation 20 ft. was located a 45x80½-ft. boiler house. On the flat close to the shore were located storage bins; these were connected by inclined conveyor with the crusher house and could chute from their water side directly into barges. The crusher plant consisted of six No. 6 and two No. 9 gyratory crushers and later a No. 21 crusher was added.

The rock was stripped of its earth overburden, the soil being washed into gullies which intersected the hillside. The rock was drilled and blasted, using first well drills and later tripod drills, and was loaded by steam shovel into 6-yd. cars hauled in train to the crusher house floor and dumped. In the blasting operations generally 60 per cent dynamite was used, the charge per hole being varied to suit local conditions of rock. Vertical, toe and breast holes were used. The vertical holes were drilled 24 ft. deep.

Quarrying and crushing were begun March 2, 1909, and crushed stone production ceased on April 30, 1913, the total production of the plant having been 1,921,579 cu. yds. of crushed stone. On Jan. 1, 1910, exact cost keeping was begun and the cost of quarrying at Porto Bello from that date to the close of operations on April 30, 1912, is given in Table III

TABLE III.

Item.	1909-10.	1910-11.	1911-12.
Cubic yards	404,924	864,033	440,113
Stripping	$0.0360	$0.0174	$0.0019
Drilling	.0875	.0450	.0431
Blasting	.2949	.1980	.1622
Loading	.1596	.0921	.0771
Transportation	.1086	.0758	.0669
Tracks	.0601	.0464	.0346
Power	.0417	.0289	.0353
Maint. equipment	.0776	.0981	.0621
Plant arbitrary	.3401	.3350	.4897
Total	$1.2091	$0.9367	$0.9747

The cost of crushing the quarried rock for the same period is given in Table IV

TABLE IV.

Item.	1909-10.	1910-11.	1911-1
Cubic yards	404,924	864,033	440,41
Crusher operation	$0.0559	$0.0355	$0.02
Stone bins and conveyors	.0413	.0311	.01
Power	.0481	.0394	.04
Maint. equipment	.0573	.1158	.08
Plant arbitrary	.1851	.1726	.25
Total	$0.3777	$0.3944	$0.42

Summarizing the quarrying and crushin costs we get for cost of stone production a Porto Bello the following:

Year.	Per cu. y
1909-10	$1.40
1910-11	1.33
1911-12	1.59

These costs do not include any charge fo general administration.

For the year ending June 30, 1911, and th succeeding ten months ending April 30, 191 records were kept of crushing plant perform ance with the results presented in Table V.

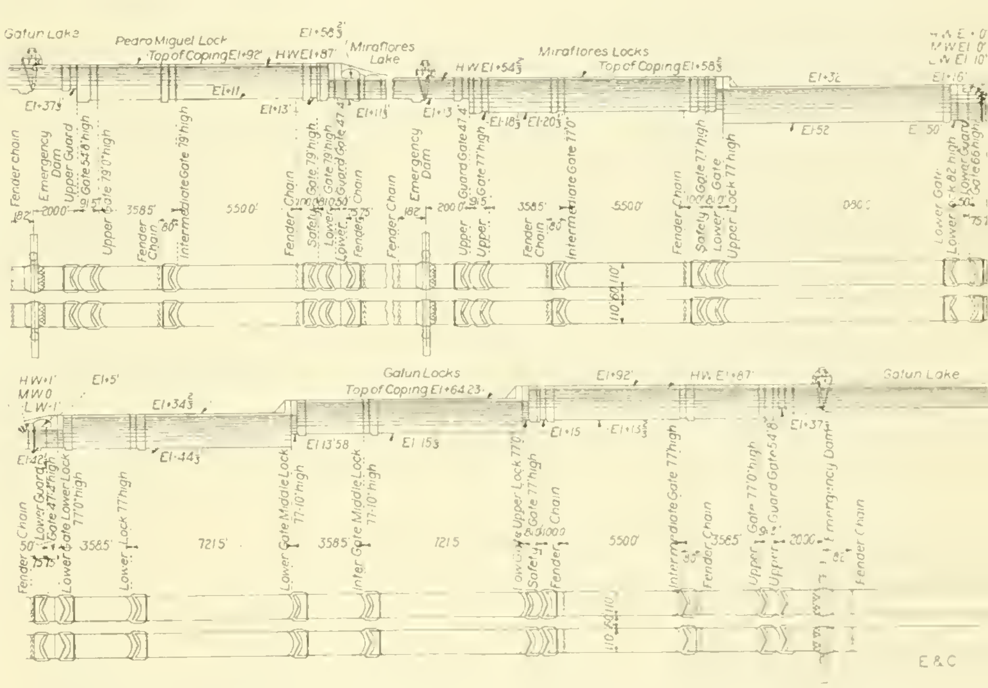


Fig. 18. Diagram Plans of Panama Canal Locks Showing Locations of Gates, Fender Chains, Emergency Dams.

TABLE V.

Item.	1910-11.	1911-
Hours under pay, per cent	100	100
Hours working, per cent	72.66	49.8
Delays, per cent	27.34	50.1
Repairs to crusher	1.84	0.5
Repairs to cross conveyors	2.93	0.5
Repairs to bin conveyors	2.58	0.5
Waiting for stone	7.25	30.6
Crusher choked	1.17	0.5
Waiting for barges	5.93	13.6
Other delays	5.64	3.7
Totals	27.34	50.1

Waiting for stone and waiting for bar were the chief causes of delay.

The stone from the bins was chuted i steel barges of 700 cu yds. capacity and tow to Colon Harbor and then inland through old French canal and a dredged channel unloading docks on the East Diversion, n Gatun. Barges tied up to the west wh were unloaded by duplex cableways wh transported the stone to a 200,000 cu. yd. st pile. The cost of towing and unloading st from Port Bello was for the period Jan. 1910, to April 30, 1912, as given in Table

TABLE VI.

Item.	1909-10.	1910-11.	1911
Towing, cu. yds	404,924	864,033	440,
Operation of tugs and barges	$0.1660	$0.1592	$0.
Maint. equipment	.0666	.1063	.
Plant arbitrary	.2240	.1975	
Totals	$0.4586	$0.4330	$0.
Unloading:			
Cableway operation	$0.1697	$0.1168	$0.
Maint. equipment	.0692	.0929	.
Power	.0166	.0195	.
Plant arbitrary	.1731	.2050	.
Totals	$0.4268	$0.4342	$0.5

A final summary of the cost of stone production, including general administration charges, is given in Table VII.

TABLE VII.

Item.	1909-10.	1910-11.	1911-12.
Quarrying	$1.2091	$0.9307	$0.9747
Crushing	.3777	.3944	.4264
Towing	.4586	.4630	.4618
Unloading	.4268	.4342	.5260
Division expense	.0854	.0551	.0269
Totals	$2.5576	$2.2774	$2.0158
Official totals	2.6283	2.3403	2.4952

The official totals of cost of all crushed stone in storage at Gatun include a considerable yardage transported by rail to storage, for which there was an additional charge. The performance of the unloading cableways at Gatun for the two years ending June 30, 1912, is given in Table VIII.

TABLE VIII.

Item.	1910-11.	1911-12.
Cu. yds. handled	742,408	346,767
Cu. yds. per strand per hour worked	42.77	52.74
Cu. yds. per strand per hour under pay	21.14	30.42
Hours under pay, per cent	100	100
Hours delay, per cent	50.57	42.31
Hours working, per cent	49.37	57.69
Delays		
Repairs	10.97	5.70
Electrical	3.17	1.85
Waiting for barges	24.21	26.35
Moving barges	....	2.09
Other delays	12.22	6.32
Total	50.57	42.31

Besides the cableways from two to five boom derricks were operated unloading stone and sand. The performance of these derricks for the two years ending June 30, 1912, is summarized in Table IX

TABLE IX.

	1910-11. Hours.	1910-11. Pct.	1911-12. Hours.	1911-12. Pct.
Time delayed	11,694	50.92	3,677	48.73
Time working	11,274	49.08	3,869	51.27
Time in operation	22,968	100.00	7,546	100.00

Pacific Locks.—Crushed stone for the concrete work at Pedro Miguel and at Miraflores was obtained from one quarry opened in the west face of Ancon Hill. The rock was drilled and blasted and loaded into cars by steam shovel. The cars ran from the quarry face by a series of switch-back tracks to the crusher house and onto the crusher floor. The crusher plant was a side hill plant and consisted of a No. 12 gyratory crusher feeding to four No. 6 gyratory crushers. The crushed stone was conveyed by horizontal belt conveyor to storage bins mounted on trestles over railway tracks. Cars loaded from the bins were hauled in trains to Pedro Miguel and Miraflores and the stone was dumped from trestles into storage piles parallel to the locks and close enough to be within reach of the berm cranes described elsewhere.

Operations at Ancon quarry began Feb. 10, 1910; detail operations are available from this date to July 1, 1912. For the year ending June 30, 1913, the output of crushed stone was 688,301 cu. yds., which cost in storage piles at the locks 77.95 cts. per cubic yard. The cost for quarrying, crushing and transporting stone for the Pacific locks to July 1, 1912, is given in Table X.

TABLE X.

Item.	1909-10.	1910-11.	1911-12.
Quarrying in cu. yds.	175,174	855,824	839,279
Stripping	$0.1736	$0.0476	$0.0423
Drilling	.1388	.0449	.0552
Blasting	.0927	.0421	.0465
Loading	.0689	.0152	.0356
Transportation	.1402	.1727	.0576
Tracks	.1741	.0213	.0379
Maint. equipment	.0855	.0333	.0568
Plant arbitrary	.2300	.2147	.1903
Total	$1.1038	$0.5548	$0.5222
Crushing:			
Crusher operation	$0.0346	$0.0172	$0.0179
Bins and conveyors	.0089	.0045	.0045
Power	0528	.0183	.0187
Maint. equipment	.0033	.0175	.0311
Plant arbitrary	.0649	.0762	.0592
Total	$0.1645	$0.1337	$0.1314
Transportation:			
Train operation	$0.0728	$0.0447	$0.0366
Track	.0209	.0075	.0004
Dumping	.0138	.0098	.0110
Maint. equipment	.0310	.0157	.0206
Plant arbitrary	.0500	.0553	.0590
Total	$0.1885	$0.1330	$0.1276
Division expense	.0455	.0228	.0181
Grand total	$1.5023	$0.8443	$0.7996

Besides furnishing stone for the lock concrete, the Ancon quarries shipped crushed stone to other divisions and for municipal work. The performance of the Ancon crushers for 1910-11 and 1911-12 is shown by Table XI.

TABLE XI.

Item.	1910-11.	1911-12.
Yardage crushed	855,824	937,908
Time working, pct	74.20	71.91
Time delayed, pct	25.80	28.09
Delays:		
Repairs	5.11	6.43
Crusher choked	4.67	2.86
Waiting for stone	3.15	10.55
Waiting for trains	3.38	3.79
Other delays	9.49	4.52
Total delays	25.80	28.09

SAND SUPPLY.

No local supplies of sand ample in quantity and of good enough quality for concrete were available near the lock and dam sites. All sand was dredged, loaded on barges and shipped by water and railway to the various places where it was needed. The three main destinations for sand were Gatun, for the lock and spillway works, and Pedro Miguel and Miraflores, for the locks.

Gatun Locks.—Sand for the Gatun locks and spillway was secured at Nombre de Dios, about 40 miles from Colon; it was obtained

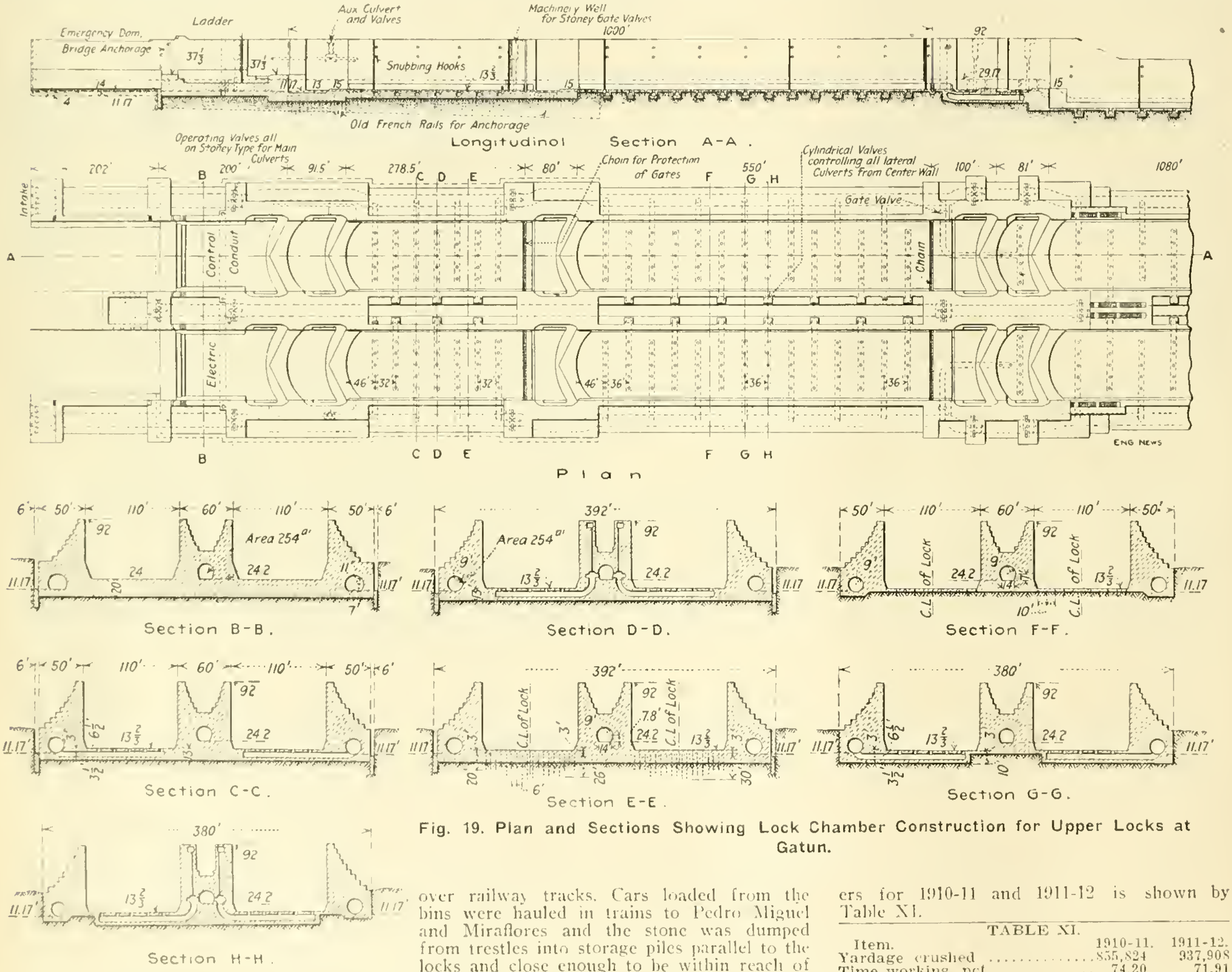


Fig. 19. Plan and Sections Showing Lock Chamber Construction for Upper Locks at Gatun.

by dredging into the beach from the sea. Work was begun developing the supply on Oct. 22, 1908, and during the period to July 1, 1909, besides development channel constructed, a plant was installed consisting of a remodeled old French suction dredge, a lighter carrying a derrick operating a clamshell, and locomotive cranes. There was also provided a fleet of barges and tugs. To July 1, 1909, 12,065 cu. yds. of sand were shipped, the barges, like the stone barges from Porto Bello, going up the old French Canal to the unloading wharves at Gatun. The sand was unloaded from barge to stock pile by cableways and derricks, the performance of which is given in Tables VIII and IX. The cost of sand supply from Nombre de Dios by general items was for the period of record as follows:

Item.	1910.	1911.	1912.
Production	$0.82	$0.85	$1.11
Towing	0.62	0.54	0.63
Unloading	0.35	0.36	0.41
Total	$1.91	$1.86	$2.24

The totals in this tabulation are modified totals to allow for certain rail and general charges.

Pacific Locks.—Sand for a lock construction at Pedro Miguel and Miraflores was dredged from a bay formed by Point Chamé, which is about 20 miles up the coast from Balboa. It was dredged by a French self-propelling ladder dredge, loaded into barges of 500 cu. yds. capacity and towed to Balboa where it was transferred into storage bins by means of rapid unloading cranes. It was chuted from the bins into cars and hauled by train to the storage piles at the lock sites. The cranes used were three in number and their performance records for 1910-11 and 1911-12 are given by Table XII.

TABLE XII.

Item.	1910-11.	1911-12.
Sand unloaded, cu. yds.	494,841	564,837
Time working, pct.	70.59	71.83
Time delayed, pct.	29.41	28.17
Delays, pct.:		
Repairs	2.45	0.56
Waiting for barges	7.66	0.25
Other delays	0.82	0.49
Total	29.41	28.17

The cost of sand in place in stock piles at Pedro Miguel and Miraflores for the time of record is given by general items in Table XIII.

TABLE XIII.

Item.	1909-10.	1910-11.	1911-12.
Production (dredging)	$0.2457	$0.1580	$0.1510
Towing	.2252	.2269	.1738
Unloading	.2584	.2482	.1722
Rail transportation	.2201	.1816	.1918
Division expenses	.0270	.0208	.0137
Total	$0.9764	$0.8284	$0.7025
Yardage	183,193	494,841	564,837

CONCRETE CONSTRUCTION.

Concrete construction was begun at the Gatun locks on Aug. 24, 1909, at Pedro Miguel on Sept. 1, 1909, and at Miraflores in July, 1910. Concreting on all locks was practically completed on Sept. 1, 1913. During the four years of construction the following yardages were laid at each lock at the unit costs given:

Lock.	Cu. yds.	Unit cost.
Gatun	2,067,073	$7.12
Pedro Miguel	930,090	5.51
Miraflores	1,493,600	5.02

The progress of concreting by months from the beginning to the completion of work on each lock is given by Table XIV.

TABLE XIV.

Months.	Gatun Locks 1909.	Gatun Locks 1910.	Gatun Locks 1911	Gatun Locks 1912.	Gatun Locks 1913	Pedro Miguel Lock 1909	Pedro Miguel Lock 1910	Pedro Miguel Lock 1911	Pedro Miguel Lock 1912.	Pedro Miguel Lock 1913	Miraflores Locks 1909	Miraflores Locks 1910	Miraflores Locks 1911	Miraflores Locks 1912.	Miraflores Locks 191
January	....	51,136	72,919	31,983	28,085		16,273	38,513	15,007	4,652		.	21,018	48,116	50,4
February	....	55,696	72,103	26,661	30,759		15,218	37,011	12,639	4,294		116	20,896	63,892	31,9
March	....	60,998	86,881	27,532	18,180		18,793	14,716	9,331	13,112		314	31,173	83,706	21,0
April	....	63,227	67,361	11,600	[illegible]		21,522	[illegible]	9,460	9,115		12	28,778	97,737	13,2
May	....	74,273	67,841	7,746	13,046		20,576	19,135	[illegible]	1,111		77	36,151	92,095	6,9
June	....	89,401	55,035	6,095	5,509		20,631	18,243	10,061	773		1,603	26,536	68,390	3,8
July	....	81,001	71,046	8,093	2,431		11,464	19,906	11,180	[illegible]	28	3,672	32,840	66,020	3,8
August	1,298	85,686	66,928	6,857	[illegible]	....	51,261	[illegible]	[illegible]	1,884	74	6,030	57,003	75,380	3,6
September	12,293	76,720	57,298	3,162	.	2,370	50,702	[illegible]	[illegible]			18,1[illegible]	56,083	46,120	...
October	29,378	86,949	53,636	3,252		8,210	[illegible]	21,637	6,587			22,159	60,870	54,790	...
November	30,270	75,152	43,907	6,099		[illegible]	[illegible]	19,622	7,972	....		23,871	41,720	39,870	...
December	42,833	80,212	13,590	5,697		13,007	42,831	14,360	6,958			21,533	18,770	46,740	...
Total	116,072	886,451	758,821	147,708	[illegible]	[illegible]	411,947	[illegible]	129,261	[illegible]	102	97,501	471,831	783,180	137,9

GATUN LOCKS.

Mixing and Handling Plant.—The task set for the mixing and handling plant at Gatun Locks was the construction of 2,000,000 cu. yds. of concrete in a maximum period of five years 400,000 cu. yds. per year or about 1,300 cu. yds. for each working day. Capacity and permanency were warranted. Figure 20 shows the original plant layout and it was modified only in details as the work progressed.

Cement coming to Colon by ship from the United States was transshipped to barges and towed to Gatun, where it was unloaded by crane into the cement storehouse. Stone from Porto Bello and sand from Nombre de Dios came to Gatun in barges and was unloaded by cableways into stock piles. Cars passing through tunnels under the cement shed and stock piles received the cement, sand and stone in proper proportions and proceeded to the mixer house where they were emptied into the charging hoppers over the mixers and then proceeded around a loop and returned to the stock piles for another trip. The mixers discharged into cars which traveled on tracks parallel to the locks. The buckets of concrete were lifted from the cars by cableways which spanned the lock chambers and which carried the buckets to the forms and discharged them.

The unloading cableways comprised two duplex and one single cableway operating grab buckets; the performance of these cableways and the cost of unloading stone by them are given in Tables VIII and VI. The concrete handling cableways, four in number, were all duplex cableways. All cableways were mounted on traveling towers and could move along the work as occasion demanded. The performance figures of the lock cableways for the two years of record are given in Table XV.

TABLE XV.

Item.	1911.	1912.
Strands operated, number	8	8
Yardage placed, cu. yds.	616,661	309,534
Handling concrete, per unit	64.16	38.74
Handling steel and forms, pct.	8.80	17.77
Total time working pct.	72.96	56.51
Repairs to cableways, pct.	3.44	1.27
Waiting for forms, pct.	13.79	30.12
Moving, pct.	4.81	6.12
Other delays, pct.	1.20	....
Total time delayed, pct.	27.04	43.49
Yards per strand per hr. working	32.45	31.46

The main mixer plant consisted of eight 2-cu.-yd. mixers, four facing right and four left, as indicated on Fig. 20. In October, 1909, this plant was supplemented by an auxiliary steam driven plant consisting of two 2-cu.-yd. mixers and located near the south ends of the locks. The performance figures of these mixer plants for the two years of record is indicated by Table XVI.

TABLE XVI.

Item.	Main Plant. 1911.	Main Plant. 1912.	Auxil'ry Plant. 1911.	Auxil'ry Plant. 1912
Aver. No. mixers worked	6.08	4.30	2	1.89
Yardage mixed	602,851	343,364	226,476	80,511
Time w'ked, pct.	58.74	43.07	59.43	32.39
Delays, pct.	41.26	56.93	40.57	67.61
Repairs	6.77	.....	0.89	0.35
Waiting for cars	33.34	56.93	37.86	66.30
Other delays	1.15	.....	1.82	0.96
Yds. mixed per hr. working	45.22	59.26	68.37	68.54

The plant as described was used throughout the work except for minor alterations; the cableways were strengthened and several details of the cars and railway from stock piles to mixer were modified. The entire plant was operated by electric power generated by a steam turbo generator station built near the lock site. This plant also furnished current for lighting and miscellaneous power purposes.

Wall and Culvert Forms.—In planning fo the laying of concrete decision was reache to use movable steel forms as far as practicable, not only for the main and lateral culverts, but for the lock walls themselves. Th walls were laid in monoliths, 36 ft. long and the forms for these monoliths were assembled for the entire height of the wall, except that the backs of the forms were of necessity built up as the concrete rose, due to th narrowing of the wall toward the top. Th front of the wall was formed against a built up steel plate, which was supported by a stee tower so designed as to withstand the horizontal thrust of the concrete, and provide with jacks, by means of which the plate coul be kept accurately adjusted in position. Th tower rested on rails, and could be readil moved from one position to another in layin the successive monoliths. The sides of th form were built up and bolted to the fron form and braced to withstand the thrust o the concrete. In any one position of a cableway there were forms ready for monolith in the two side walls and in the middle wal and when one set of cableways begin to la concrete, the interruption usually due to for construction was obviated as far as practicable. Drawings showing the general features of the forms and towers are given b Fig. 21.

Wood forms were employed in placing th floor concrete and for certain irregular details.

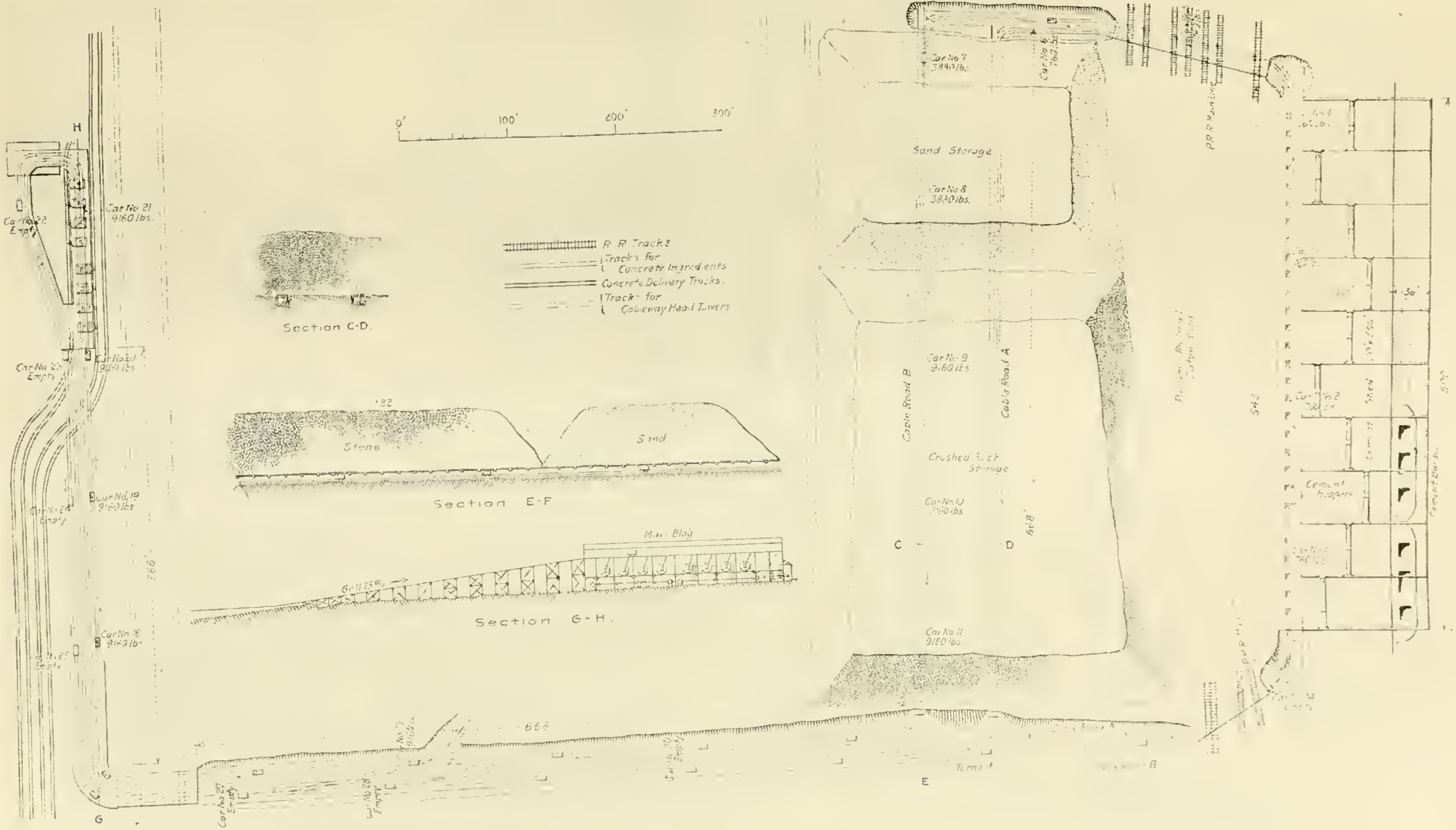
The culvert forms consisted of a horsesho shaped form for the main center wall culvert, circular forms for the side wall culvert and elliptical forms for the lateral culvert All culvert forms were of steel and were designed to be collapsible. Like the wall form they were used over and over and were handled from place to place by the lock cableways.

Concrete Costs.—Concreting was begun Gatun on Aug. 24, 1909. The concrete wa a 1:3:6 mixture with a facing of 1:2:4 concrete around main culverts and a 1:3 mixtur for finished horizontal surfaces. Rubble ston were embedded in all mass concrete wor Detail cost records are available from Jan. 1910, to July 1, 1912, and are given in Tab XVII.

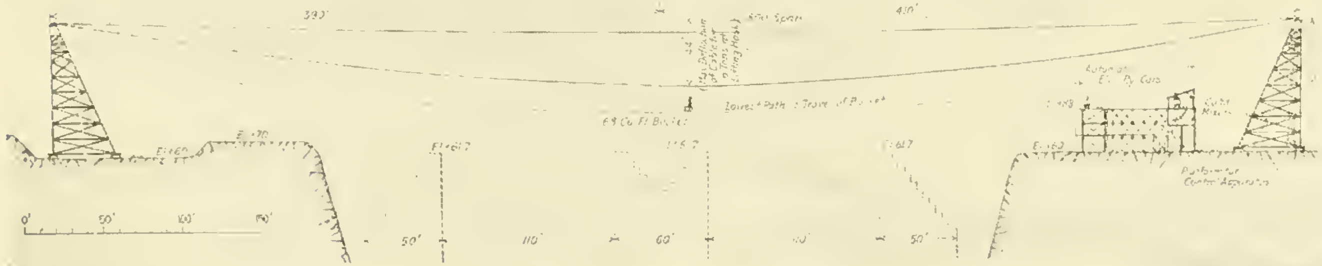
TABLE XVII.

Item.	1910.	1910-11.	1911-1
Concrete, cu. yds.	386,946	829,317	376,41
Cement	$1.9786	$1.7784	$1.53
Stone	2.3038	2,0960	1.99
Sand	0.7602	.8614	1.01
Mixing	0.2438	.1756	.17
Total	$5.2864	$4.9114	$4.72
Large rock, cu. yds.	10,785	73,609	11,1
Cost	$3.0652	$1.3366	$1.32
Masonry, cu. yds.	397,731	902,926	390,61
Concrete	$5.1431	$4.5111	$1.54
Large rock	.0831	.1090	.04
Wood forms	.....	.3802	.50
Steel forms	.4952	.1067	.15
Placing	.4285	.3285	.47
Reinforcement	.0111	.0241	.01
Pumps	.0332	.0417	.08
Power	.0726	.0666	.08
Maintenance equipment	.1971	.2357	.21
Plant, arbitrary	.6035	.6969	.89
Division expenses	.1051	.0914	.10
Administration	.3523	.2208	.26
Total	$7.5245	$6.8127	$7.39

Previous to Jan. 1, 1910, there were place in the Gatun locks 116,072 cu. yds. of co crete at a cost of $8.54 per cubic yard. Between June 30, 1912, and July 1, 1913, 137 218 cu. yds. were laid at a unit cost of $6.5 All the above costs are for plain concrete. In addition there were laid considerable yar

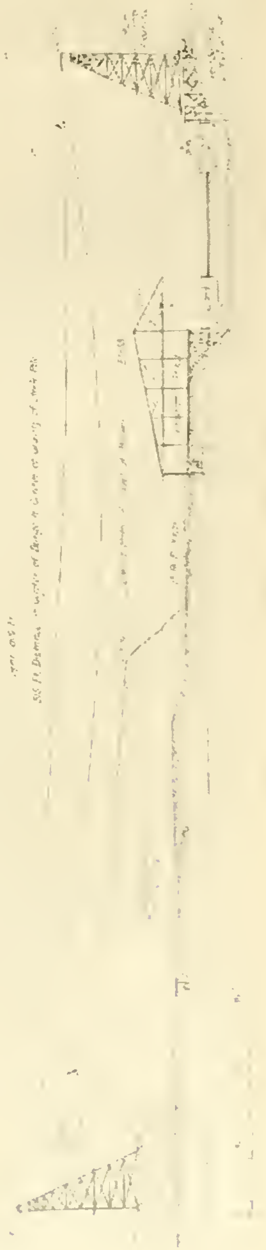


GENERAL PLAN OF PLANT FOR ASSEMBLING CONCRETE MATERIALS


393'
800' Span
410'
Max. Deflection of Cable for 6 Tons at Lifting Hook
Lowest Path of Travel of Bucket
68 Cu. Ft. Bucket
El. 70
El. 87
El. 63
0' 50' 100' 150'
50'
110'
60'
50'

CROSS-SECTION OF LOCK PIT. SHOWING CABLEWAYS.

THE
CONCRETE MAKING PLANT
AT THE
GATUN LOCKS OF THE
PANAMA CANAL



UNLOADING CABLEWAYS AND CEMENT HANDLING PLANT

Fig. 20. General Plan of Construction Plant for the Gatun Locks.

ages of reinforced concrete of which records are given as follows:

Year.	Yardage.	Per cu. yd.
1910-11	8,211	$13.58
1911-12	59,883	12.66
1912-13	37,532	10.58

The average cost given above for all concrete in the Gatun locks includes this reinforced work and all miscellaneous high cost finishing work.

PACIFIC LOCKS.

Entirely similar plant, and to a large degree the same plant, was used in constructing the locks at Pedro Miguel and at Miraflores.

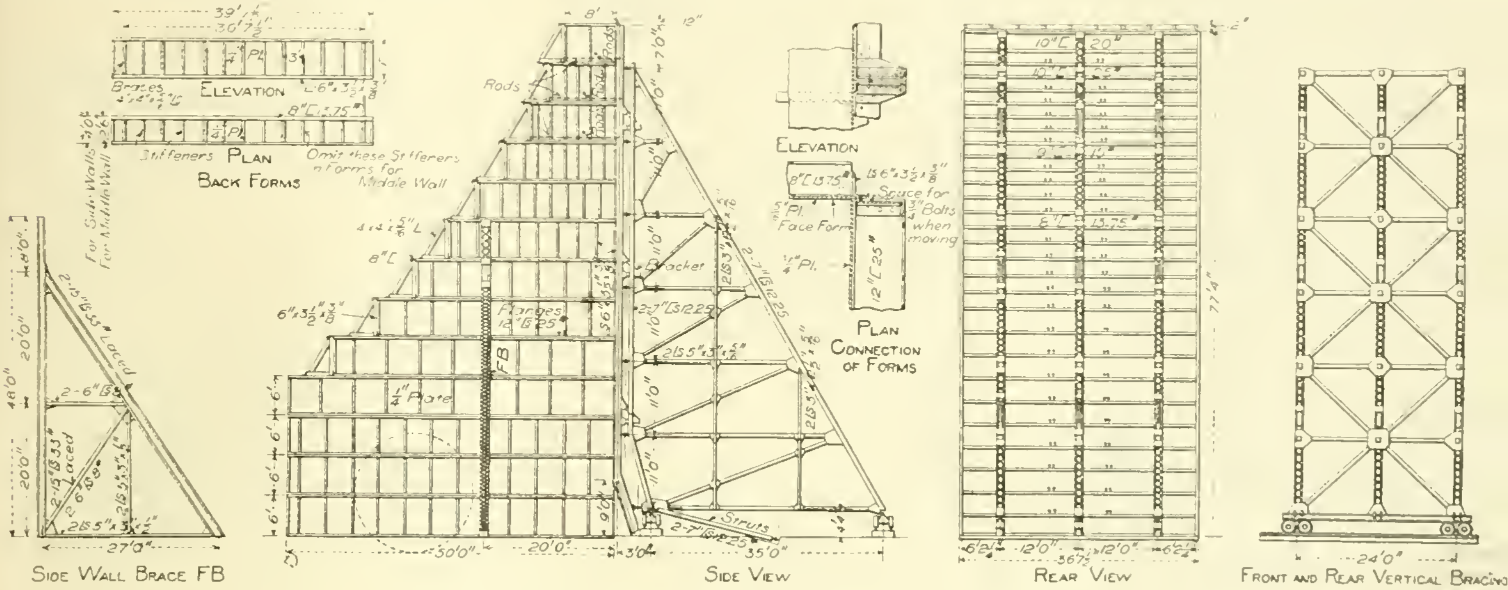


Fig. 21. Steel Forms for Wall Construction at Gatun Locks.

This plant consisted of eight cantilever cranes, four each of different types, designated, respectively, as Berm and Chamber cranes. These machines. Figs. 22, 23 and 24, differently arranged, were used for placing concrete at both Pedro Miguel and Miraflores. The high and unstable banks flanking the former site made it necessary to develop the handling plant in a longitudinal direction, by placing the mixing machinery and storage in the forebay, introducing a narrow-gage road for transporting the mixed concrete to the placing machines located in the lock chambers. At Miraflores the handling of concrete was materially simplified as the nature of the ground admitted of placing the storage piles and mixing machines on both sides of the lock pit, thus dispensing with the narrow-gage road.

Pedro Miguel Plant.—The essential features of the installation at Pedro Miguel were: (1) Storage trestles, for unloading and storing sand and stone; (2) berm cranes, for handling raw material and mixing concrete; (3) narrow-gage road, for transporting concrete, and (4) chamber cranes, for placing concrete.

In the forebay of the locks framed trestles were erected on both sides of and parallel to the canal axis; each had an average height of 28 ft. and a length of 800 ft. available for storage. Sand and crushed stone were delivered on the trestles in trains made up of 12-cu.-yd. dump cars, and the former was dumped on the outside and the latter on the inside in order to minimize the average distance of conveying the aggregate to the mixers. The total storage capacity was 15,000 and 55,000 cu. yds. of sand and stone, respectively, which would supply the mixers for 17 days of eight hours.

Two berm cranes only were used at Pedro Miguel, and were installed between the trestles on two parallel 5-ft.-gage tracks, 50-ft. centers. The cranes were entirely of steel and consisted of a tower 50x40 ft., supporting balanced cantilever arms that extended well over the storage trestles with ample clearance for a locomotive.

From the lower chords of the cantilever trusses were hung continuous trolley tracks equipped with two trolleys—one for each arm—arranged to operate 2½-cu.-yd. excavating buckets, by means of which sand and stone were taken from storage and conveyed to bins located on the crane towers. There were four bins, each of 15 cu. yds. capacity; two being used for stone and two for sand. Beneath the bins was a cement floor 2,000 sq. ft. in area on which could be stored 6,000 bags of cement. Two cement hoppers of 10 bbls. capacity each were hung from this floor.

Cement was delivered direct from box cars on an elevator attached to the tower framing and deposited by the latter on a horizontal conveyor running across the cement floor to the gratings over the hoppers, but the bags of cement could be taken off before reaching the hoppers and stored on the floor. The bins and hoppers were equipped with chutes which discharged into either one or both of the measuring hoppers that were attached to the two concrete mixers located lower down on the towers.

The mixers were 2-cu.-yd. machines operated by electric motors, and were provided with long spouts for dumping direct into buckets on the narrow-gage cars alongside. Each mixer was provided with an electrically operated recording instrument which showed graphically the time consumed in mixing each batch. The water required for mixing was supplied through automatic measuring tanks placed on the towers.

Power for operating the cranes was transmitted by an overhead line from the Miraflores power house to a substation at Pedro Miguel, where it was converted into 550-volt direct current by two 550 kilowatt rotaries and supplied to the cranes through a feeder rail supported on brackets attached to the cross ties of one of the crane tracks. All motors, except for the concrete mixers, were in a machinery house located on top of the tower truss. All operations, however, were controlled by master switches in the cabs of the trolleys, so that the operator was always immediately over and in plain view of the bucket. A 65-HP. compound-woun motor geared through two reductions to th cable drum was used for hoisting, and a 21 HP. compound-wound motor for trolleying In both of these movements dynamic brakin was used. The elevator and conveyor wer driven by a 15-HP. componnd-wound moto and each mixer by a 40-HP. shunt-woun motor. One of the hoisting motors was als used for traversing the crane by connectin through friction clutches to the traversin drums after having disconnected from th hoisting drum, the arrangements being suc that the two movements of hoisting an traversing could not be accomplished at th same time.

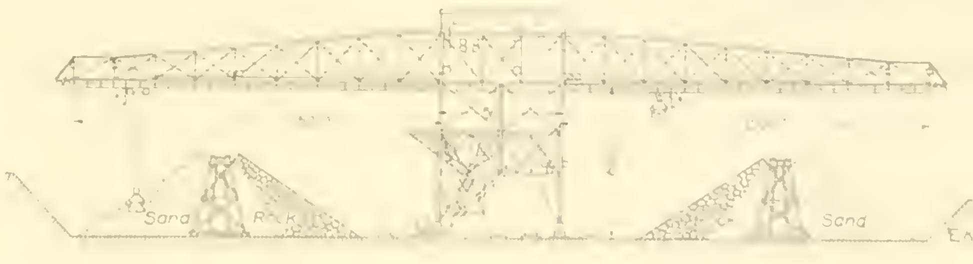


Fig. 22. Arrangement of Berm Cranes at Pedro Miguel Locks.

A narrow-gage railroad was used for trans porting mixed concrete from the berm to th chamber cranes. The difference in elevatio between the forebay and bottom of loc chambers was overcome by means of trestle with a grade of 2.3 per cent. The tracks wer laid with 70-lb. steel rails, which enabled th locomotives to attain a greater rate of spee than would be safe on the light rail usuall employed. The equipment consists of twelv 11½-ton locomotives and 21 steel fla cars, all equipped with air; each car wa large enough to hold two buckets of 64-cu.-f capacity. A train consisted of two cars an each of the latter carried a bucket so place that when alongside a berm crane the buckets could be filled from the correspondin mixers without moving the train. The train alternated in going into the respective loc chambers and stopped under the first chamber crane reached; the latter placed an empty bucket on one of the cars and picked up a loaded one from the same car; the train then moved to the next crane, where the operation of exchanging buckets was repeated, afte which it returned on the up track to the mixing cranes with the two empty buckets.

Four chamber cranes two in each lock were in service at Pedro Miguel Their func tion was to place concrete, handle forms, an the steel or cast members that were embedde in the masonry. They were steel cantilever cranes made up of a tower 56x40 ft., carried on four heavy freight-car trucks which ran on two 5-ft. gage tracks located on the floors of the locks. Each tower supported a cantilever arm 53 ft. 6 ins. long, extending over the center wall, and one 81 ft. 6 ins. long, extending over the side wall. A continuous trolley runaway 191 ft. long was suspended from the cantilevers and lower truss of the tower; the trolley was designed to run the entire length of the runway and to handle a 64-cu.-ft. concrete bucket. All operations, including opening and closing the bucket, were controlled by the operator in the trolley cab. As in the case of the berm cranes the machinery was in a house on top of the tower and was actuated by direct electric current supplied from the same source. For hoisting and trolleying 47-HP. and 21-HP. motors, respectively, were used and a 15-HP. motor for traversing the entire machine; the latter was controlled by a switch so located that the operator could reach it from his cab when in a certain position.

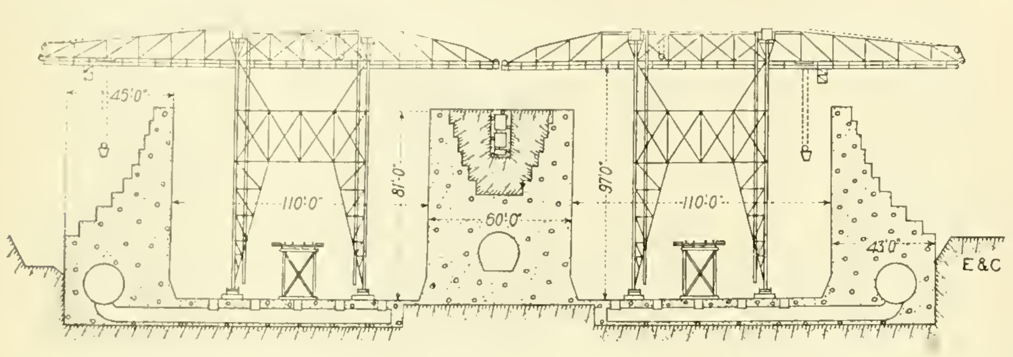


Fig. 23. Arrangement of Chamber Cranes at Pedro Miguel Locks.

Miraflores Handling Plant.—All the berm and chamber cranes used at Pedro Miguel were transferred to Miraflores locks and placed in service there between April, 1911, and April, 1912. The Miraflores plant then included: (1) Storage trestles, for sand and stone; (2) berm cranes, for handling raw material, mixing, and placing, and (3) chamber cranes, for placing.

The storage trestles were located on both sides of the lock site and under the outer arms of the berm cranes. Sand and stone were delivered in the same manner as at Pedro Miguel.

Two berm cranes on each side of the locks were used at Miraflores. They consisted of steel towers with cantilever arms extending over the storage trestles and booms 144 ft. long projecting over the side walls of the locks. Those at Pedro Miguel were dismantled and re-erected at Miraflores with the following modifications: One of the cantilever arms was removed from each and used as the outer arms for the two additional cranes, and all four were equipped with booms. The towers were equipped with bins, hoppers, and 2-cu.-yd. mixers, as at Pedro Miguel, but the operation differed in that one excavating bucket operator on the cantilever supplied the stone, while the boom carried a trolley equipped with a 64-cu.-ft. concrete bucket, into which the concrete was dumped direct from the mixers. The boom was hinged and had a vertical range of 25′ and could swing horizontally through an arc of 50 ft. It was not necessary to unhook the bucket from the boom trolley either to receive or discharge the concrete, which was deposited directly in the side walls and into a transfer hopper, from which it was discharged into the chamber crane buckets to be conveyed to the middle wall. All operations were controlled from the trolley cabs, and the motors were the same as were used at Pedro Miguel.

The chamber cranes at Pedro Miguel were dismantled upon the completion of the work there and rebuilt at Miraflores. They performed the same functions as described for Pedro Miguel.

Crane Performance Records.—From June 30, 1910, to July 1, 1912, records are available of the performance of lock cranes at Pedro Miguel and at Miraflores. Tables XVIII and XIX are compiled from these records.

TABLE XVIII.
(Berm Cranes.)

Items.	Pedro Miguel. 1910-11.	Miraflores. 1910-11.	Miraflores. 1911-12.
Concrete mixed, cu. yds.	379,190	67,774	409,651
Cranes operated, number	1.58	1.80	3.58
Mixers operated, number	3.08	3.59	6.55
Hours working, pct.	72.40	62.25	62.00
Delays, pct.	27.60	37.75	38.00
Repairs to cranes	4.44	7.23	5.09
Repairs to mixers	0.77	1.05	0.45
Waiting for material	....	1.83	....
Waiting for cars	21.39	0.09	*8.52
Waiting for forms	.....	6.16	14.04
Other delays	1.00	21.39	9.09
Total delays	27.60	37.75	38.00
Cu. yds. per hr. working	70.30	41.69	38.06

*Delay due to bridging.

TABLE XIX.
(Chamber Cranes.)

Items.	Pedro Miguel. 1910-11.	Pedro Miguel. 1911-12.	Miraflores. 1911-12.
Concrete placed, cu. yds.	403,155	28,450	234,520
Cranes working, number	3.45	1.72	2.48
Time working, pct.:			
Placing material	.....	22.75	55.34
Handling steel forms	.....	39.01	9.09
Total	67.44	61.76	64.34
Delays, pct.:			
Repairs to cranes	2.73	2.97	7.21
Waiting for concrete	9.35	8.88	11.57
Waiting for forms	13.80	12.47	14.58
Bridging	.....	4.87	4.01
Other delays	6.68	9.05	3.22
Total	32.56	38.24	46.66
Cu. yds. per hr. working	64.71	42.48	58.38

Forms.—Wood forms in built-up panels 15 ft. long and 6 ft. high were used for building the lock walls at both Pedro Miguel and at Miraflores. Each panel consisted of seven uprights 14 ft. long connected by waling pieces and lagging, the latter being placed on the upper 6 ft only; the lower 8 ft. of the uprights acted as cantilevers and were anchored to the concrete previously placed. The anchor bolts extended into the masonry about 2 ft. and were removed as the work progressed, leaving the anchor nut only embedded. The sequence of operations is indicated by Fig. 25. After the battered portion of the wall was completed, one set of forms carried it to its full height.

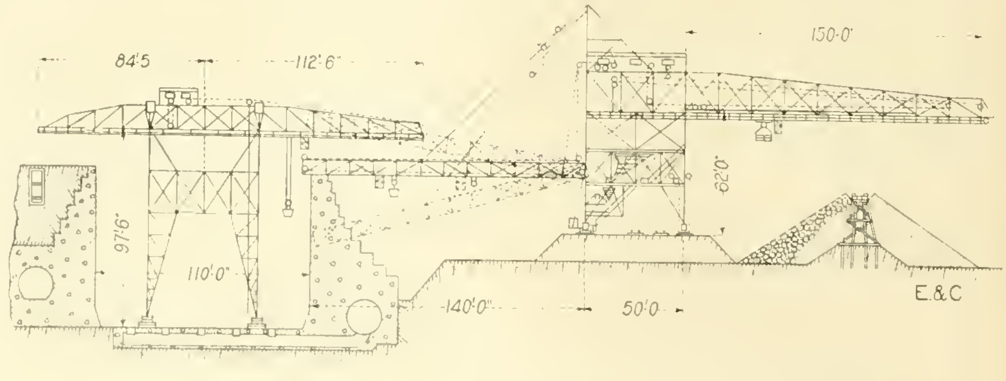


Fig. 24. Arrangement of Berm and Chamber Cranes at Miraflores Locks.

Besides the wall forms special wooden forms were used for connections of main and side wall culverts with lateral culverts and for various other details where the construction was abnormal. For the barrels of the culverts collapsible steel forms were employed.

CONCRETE COSTS.

The cost of concrete work at Pedro Miguel and at Miraflores is best considered separately for each set of locks.

Pedro Miguel.—The Pedro Miguel locks contain some 950,000 cu. yds. of concrete and placing was begun Sept. 1, 1909. Detailed records of cost from Jan. 1, 1910, to July 1, 1912, are given in Table XX.

TABLE XX.

Items.	1910.	1910-11.	1911-12.
Concrete, cu. yds.	131,354	495,037	134,193
Cement	$1.8449	$1.5365	$1.6564
Stone	1.2609	.8242	.7619
Sand	.4354	.3729	.3823
Mixing	.4559	.1771	.3562
Total	$3.9971	$2.9107	$3.1658
Large rock, cu. yds.	1,659	2,765	.....
Cost	$0.9642	$1.1483	.....
Masonry, cu. yds.	133,013	497,802	134,193
Concrete	$3.9473	$2.8945	$3.1568
Large rock	.0120	.0064	.....
Wood forms	.6808	.3904	.7596
Steel forms	.....	.0483	.0041
Placing	.5331	.3118	.5619
Reinforcement	.0028	.0367	.0074
Pumps	.0418	.0343	.0331
Power	.1055	.0454	.0199
Maintenance equipment	.0880	.1723	.1511
Plant, arbitrary	.4569	.6847	.9101
Division expenses	.0965	.0792	.1143
Administration	.3849	.2787	.3085
Total	$5.3496	$4.9827	$6.0268

Prior to Jan. 1, 1910, 33,856 cu. yds. of concrete were placed at an average cost of $7.63 per cubic yard. Between June 30, 1912, and July 1, 1913, 39,463 cu. yds. of plain concrete were placed at a cost of $6.54 per cubic yard. The cost of reinforced concrete at Pedro Miguel locks was as follows:

Year.	Yardage.	Per cu. yd.
1910-11	385	$19.75
1911-12	48,677	9.16
1912-13	18,902	9.80

The average cost given above for all concrete to July 1, 1913, includes all reinforced and other special concrete work.

Miraflores Locks.—Concreting was begun at Miraflores in July 1, 1910, and its cost in detail to July 1, 1913, is given in Table XXI. As at Pedro Miguel, there was an additional volume of reinforced concrete and the amounts and costs of this for the years of record were as follows:

Year.	Yardage.	Per cu. yd.
1911-12	22,444	$11.46
1912-13	48,185	10.80

During the year ending July 1, 1913, 402,607 cu. yds. of concrete were laid at an average cost of $5 per cubic yard. Previous to July 1, 1910, there were placed 1,630 cu. yds. of concrete at a cost of $8.11 per cubic yard.

TABLE XXI.

Items.	1910-11.	1911-12.
Concrete, cu. yds.	269,240	729,096
Cement	$1.4665	$1.7920
Stone	.8132	.7461
Sand	.3946	.3564
Mixing	.5226	.2258
Total	$3.1969	$3.1206
Large rock, cu. yds.	3,693	....
Cost	$0.7838	.....

Masonry, cu. yds.	272,933	729,096
Concrete	3.1537	3.1206
Large rock	.0106	.....
Wood forms	.4040	.4030
Steel forms	.0776	.0364
Placing	.2448	.2014
Reinforcement	.0472	.0012
Pumps	.0366	.0327
Power	.0163	.0289
Maintenance equipment	.1688	.1545
Plant, arbitrary	.1240	.5420
Division expense	.0990	.0660
Administration	.3151	.2240
Total	$4.9677	$4.8107

Lock Gate Structures.

The locks of the Panama Canal require 46 gates, distributed as shown by Table XXII.

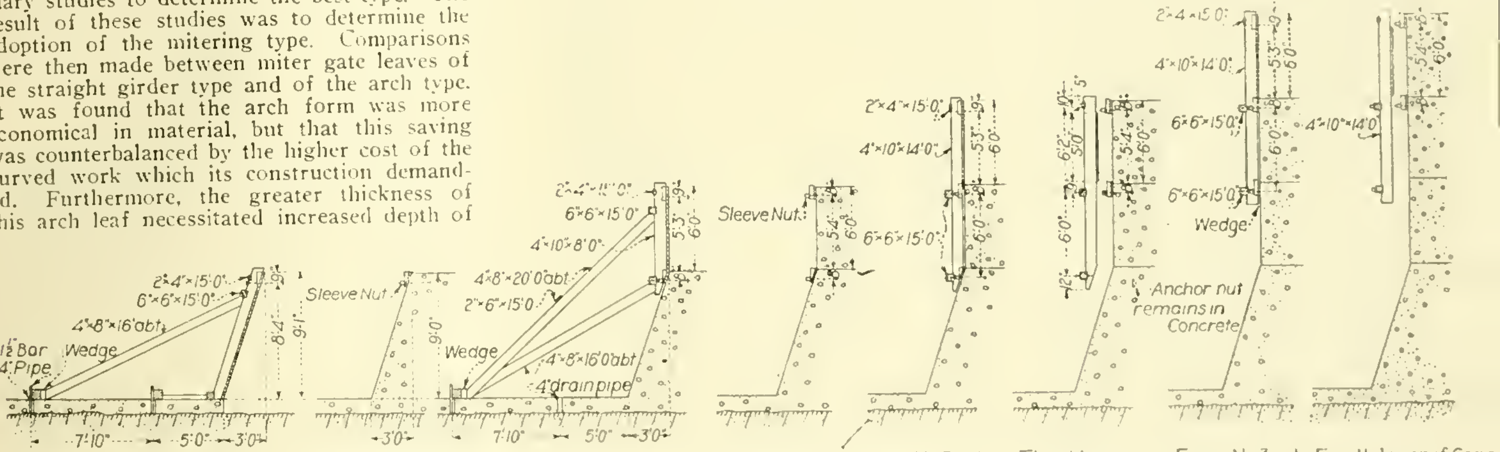
TABLE XXII.

Height.	Gatun.	Pedro Miguel.	Miraflores.	Total.
82 ft.	..	..	2	2
79 ft.	..	8	..	8
77 ft.	16	..	8	24
66 ft.	..	..	2	2
64 ft. 8 ins.	2	2	..	4
47 ft. 4 ins.	2	2	2	6
Totals	20	12	14	46

All gate leaves are of the same width, and they are all alike structurally. The design adopted was preceded by exhaustive preliminary studies to determine the best type. The result of these studies was to determine the adoption of the mitering type. Comparisons were then made between miter gate leaves of the straight girder type and of the arch type. It was found that the arch form was more economical in material, but that this saving was counterbalanced by the higher cost of the curved work which its construction demanded. Furthermore, the greater thickness of this arch leaf necessitated increased depth of gate recesses in the side walls and consequently thicker walls. Many other matters were given separate study. The selection made was a girder gate leaf of the general design shown by Fig. 26.

The leaves, as stated above, vary in height, but are uniformly 7 ft. thick and 65 ft. wide. Their frame or skeleton consists of horizontal girders spaced from 3⅔ ft. to 5 ft. apart; of vertical frames which extend the entire height of the leaves; of intercostals which stiffen the sheathing plates at intervals of about 2 ft.; and of two vertical diaphragms for transferring the thrust of the leaves along the quoin and miter posts to the web plates in the horizontal girders. Sheathing doubling and cover plates are riveted to the upstream and downstream sides of the main frame, and the ends are closed by flat plates connected to the skeleton by continuous bent plates. The lower part of the gate forms an air chamber sub-divided into three water tight compartments, while the upper part is arranged to permit the water to flow in and out freely. To the vertical ends of the leaf are attached steel reaction castings carrying the miter and quoin bearings, arranged as shown by Figs. 27 and 28. A full description of these castings will be given in the section on lock gate erection.

The gate leaves as described weigh up to 1,183,700 lbs. for the heaviest. They are hinged at the top by a pin, as shown by Fig. 29, and rest at the bottom on a hemispherical pintle, Fig. 30. In designing hinges and leaves assumptions were made of the most disadvantageous conditions of stress and the steelwork was designed to meet them. For example the assumption was made that the sill bearing alone or the miter bearing alone held the leaf against total pressure instead of this pressure being divided between miter posts and sills.



Form No1 set up for first Layer of Concrete

First Layer of Concrete in Place and form No1 removed

Form No.2 set up for Second layer of Conc

Second Layer of Conc in place and form No2 removed

Form No3 set up for third Layer of Concrete

Third Layer of conc. in place form No3 pulled off wall ready to be lifted

Form No3 set up for fourth layer of conc

Fourth layer of Conc. in place Form No.3 pulled off wall ready to be lifted

Fig. 25. Diagram Showing Operations in Using Wall Forms at Pedro Miguel and Miraflores Locks.

LOCK GATE ERECTION.

The method of erecting the lock gates was planned with care and all operations were conducted with exceptional precaution. This attention was warranted by the number of gates to be erected and was also demanded by the importance of accurate hang and fit to ease of operation, proper distribution of stresses and watertightness. Erection plans were devised by the contractor and approved by the Isthmian Canal Commission. They involved three sets of erection processes besides the transportation to the locks and the distribution of materials at the locks.

The horizontal girders, some 1,500 in all, each 65 ft. long and weighing about 18 tons, with their stiffening angles and beams, were riveted up complete in the United States and together with the vertical frames, diaphragms, intercostals and plating, were sent by ship to Colon and transferred to train which delivered them close alongside the locks. The service track arrangements at the lock are indicated by Fig. 31. It is noted that the track nearest the lock chamber is connected by curves to tracks passing onto the erecting bridges which spanned the lock chamber as indicated by Fig. 32. There were provided eight erection bridges so that erection of 16 leaves simultaneously was possible. The gate parts arriving on train were unloaded onto the fill behind the side walls by means of locomotive cranes of which ten were employed. These cranes also picked up the gate parts, carried them onto the erection bridge and lowered them into place in the gates being erected.

Erection proper began in the case of each leaf with the placing of the bottom girder. The socket casting which sets over the pintle, the large stud casting just above this girder, and the reaction frame at the miter end were riveted and bolted to the girder in the storage yard and all necessary caulking was done. Two cranes were required to handle these girders, which, with the attached posts just noted, weighed 21 tons for the high gate. One crane handled all other gate posts. The bottom girders were lowered onto a platform of I beams supported partly by the gate sills and partly on small temporary pedestals of concrete. On this platform the girder set at an angle of 15° with the lock wall and with its wall end about 1 ft. from the pintle the platform was just high enough to permit the socket casting to clear the pintle when the finished leaf was rolled into position for hanging. When the bottom girder was in place the remainder of the leaf framing was built up on it, the vertical frames and diaphragms being attached in succession befor the successive horizontal girders were lowered into place. The intercostals were generally attached later.

Special precautions were taken to insur that the framework was true in shape a erected. A special machine was installed a the shops where the gates were fabricated by which ends of girders were planed simultaneously. In the field fine steel wires wer used to check the shape of the skeleton a erection proceded, and all observed error were corrected at once. To insure correc total height adjustable pillers were used between diaphragms and the girders at the top the fourth panel and the bottom. As the result of this care in the erection it is believed that the greatest deviation from plumb line in any of the gate frames did no exceed ⅛ in.; and that the heights of al gates agree within less than ¼ in. with th figures shown on the plans.

Generally the erection of the framewor was completed before the sheathing was begun, but in any case all riveting of the framework was completed before the sheathing wa entirely in place. The manner of performin the sheathing was to lower the plates into position and temporarily bolt them in place be finally riveted all in one consecutive o eration. This plan made it possible sooner release the erecting bridge and move it another gate.

The second of the three sets of erection o erations was the exact fitting and riveting the sheathing. In all some 4,000,000 rive had to be driven and some of the larg leaves had 55,000 rivets each. All holes i these rivets had to be reamed in the fie The first step was to adjust the plates a very firmly to bolt them in position. T rivet diameters were ⅞ in. and 1 in., and the shops the holes were all punched or drill smaller than 11/16 in. All the more accessi rivets were reamed by special electrica driven reamers, of which 16 were used; holes in curved parts and other holes di cult to reach were reamed with small ha reamers driven for the most part by co pressed air. Riveting and caulking were p formed by standard pneumatic machines a all for no particular mention.

The third step in the erection process w the attachment and readjustment of the action castings and bearing plates. This w was a somewhat intricate process of care fitting, and the following description of process in detail is quoted from the engine report of work done in 1912:

> Along both the quoin and miter posts m lic bearings are used in place of timber. He steel castings are embedded in the concrete the hollow quoins and similar castings are tached to the ends of the gates by bolts a rivets. In order to obtain an exact bear and also to provide for easier renewal, s

arate bearing plates are fastened into machined recesses in these castings. The hollow quoin castings were set in the concrete during the construction of the walls, and many of them show considerable deviation from an absolutely correct position. The erection and adjustment of the bearing plates formed a part of the gate contract and preceded the erection of the gates themselves. The plates are attached to the castings by 1 in. bolts placed 17 ins. apart. Their exact adjustment is made by these bolts and by temporary stud bolts placed at intervals of about 2 ft. along the center line of the plates. A fine steel wire is fastened at the top and the bottom of the quoin so as to pass exactly through the center to which the curved surfaces of the bearing plates are to be set. These wires are fixed in position by reference to a plumb line placed directly over the center of the pintle below. By using a flat metallic template, which is curved on one side to the correct radius and notched at the other to indicate the center, the several quoin plates are readily revolved about their axis and shifted forward and backward into correct positions. Even with the errors of as much as $^3/_8$ to $^1/_2$ in. in position of the castings there has been no difficulty in setting the bearings exactly, although a small amount of grinding was at times necessary. Extreme care has been taken to get the best results and it is believed that the deviation in the bearing surfaces rarely exceeds 0.004 to 0.005 in. from an absolutely correct position.

After plates are exactly aligned, melted babbitt is poured into the spaces between the plates and the castings through some of the bolt holes for adjustment. These are used as pouring holes at intervals of about 4 ft. After a little practice, no difficulty was experienced in pouring the metal successfully. A rather large melting pot is desirable, and a high temperature. To avoid explosions from dampness, it has proved necessary to blow out compressed air into the spaces to be filled just before pouring. The joints between the bearing plates and castings are finally calked. The castings along the gate leaves are attached and secured to same before the gates are moved.

In order to insure a good bearing between the castings and the end plates on the gate frame, the latter are carefully ground with emery wheels. No attempt is made to obtain an absolutely true plane for the whole height of the leaf, as minor errors are readily corrected by the adjustment of the bearing plates. The deviation from a true plane is, however, very small. The grinding has for its purpose the obtaining of good contact between the individual castings, which are a panel in height, and the end plates. An exact contact is insisted upon along the outer edges of each casting, and also opposite the reaction diaphragms, but small depressions at other parts of the surface are permitted.

The adjustment of the wedge-shaped plates between the castings and the sides of the structural frame is somewhat difficult, and many of the bolt holes have to be reamed from the inside of the leaf. After the leaf is in correct position on the pintle with the bearing plates loosely attached to the reaction castings, the final adjustment of the plates is made. For the quoin end this is done by placing the leaves in their correct position against the sill and then forcing the several bearing plates against the fixed plates in the hollow quoin. This is done by means of the bolts which fasten the plates to the castings and of small temporary bolts or jacks attached to the ends of the permanent bolts and bearing with their other ends against the opposite surface of the casting. With the plates thus firmly held, the gate is revolved into a wide-open position and the plates babbitted through holes in their faces. Figures 27 and 28 show these bearings. It may be noted that there is sufficient room for adjusting the bolts when the leaves are closed, and also that the bearing is accessible for pouring the babbitt and for the possible future renewal of the bearing plates when the leaves are wide open.

A very close adjustment has, in the manner just mentioned, been made in the case of the first two gates so far finished. It should be added that the leaves are always adjusted in the early morning, when the temperature is lowest and the gates are at a uniform temperature in all their parts. It was expected to adjust the bearings at the miter posts in the same way by bringing the two leaves into a correct closed position and pressing the bearing plates against each other by means of jacks. The variations in the lengths of the leaves from temperature changes, even within a very short period of time, have made it necessary to abandon the method. It also proved difficult to hold the tops of the leaves in a fixed position with reference to each other. The method which seems to give the best results is to close the leaves, bring the plates at the top and bottom into contact, and, after fixing them into that position, to open the leaves and adjust all the bearings on each miter post to a true line between the top and bottom. This is done by the use of the fine wires and the adjusting bolts, and, after babbitting, the gates are brought together, so that irregularities may be observed and corrected by grinding. In order to counteract slight possible drooping of the leaves it may prove best to use a small spacer between the two bearings at the top when first adjusting the top plates at the miter posts.

The adjustment of the timber at the sill is made after the miter and quoin bearings are completely finished. These timbers rest in castings with a space about 1 inch wide for grouting. Hence they can readily be pressed against the gate leaves when they are closed, bolted fast, and afterwards grouted.

The final work is the testing of the leaf for water-tightness. In order to do this, it is blocked up securely and the entire air chamber filled with water, the manshaft being also filled to the top of the leaf, giving a head at the bottom equal to the full height of the gate. Any leaks along the calked seams or at the rivets are at once corrected by calking. The tightness of the bulkheads subdividing the air chambers is afterwards tested by successively emptying the several compartments, thus exposing the bulkheads to one-sided pressure.

After the gates were erected they were moved into position over the pintle and carefully "hung." The appliance for moving the erected leaves is illustrated in Figs 32 and 33. It consisted of two steel frames erected on opposite sides of a leaf and attached to the leaf by temporary bolts. These frames stood on pedestals which rested on rollers. A wedge device in the pedestals permitted the frames to be elevated until the leaf suspended between them hung clear off the erecting

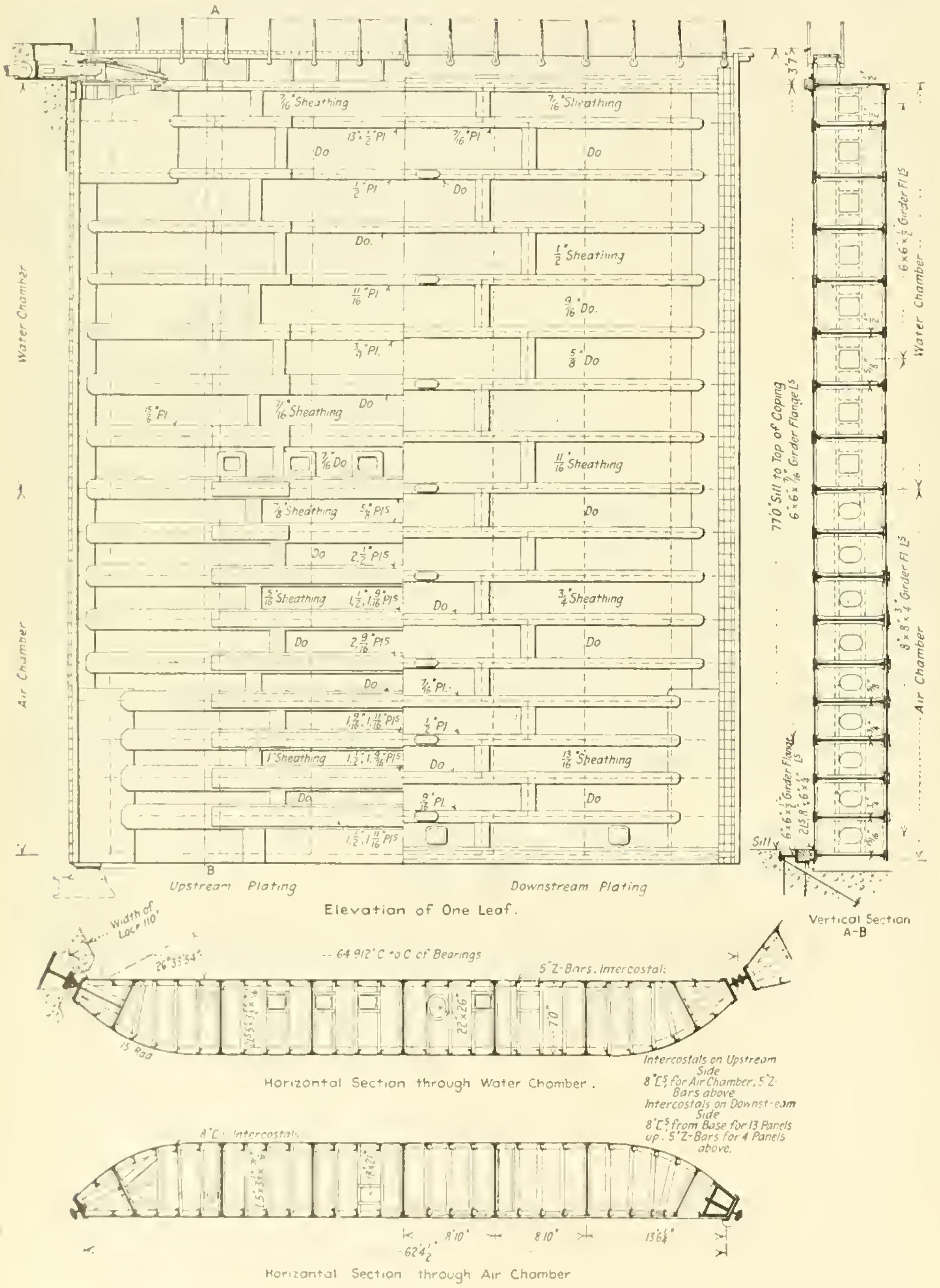
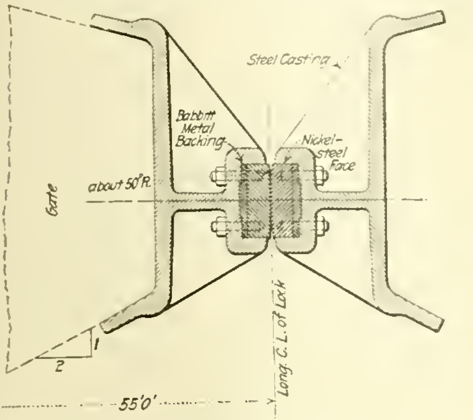


Fig. 26. Elevation and Sections of Leaf for Mitering Lock Gates.

platform. The frames were then moved on their rollers until the leaf was in position, when it was lowered onto the pintle and the top hinge was placed. Hydraulic jacks furnished the power for moving the frames and leaf; about two days were required for the operation.

LOCK GATE ERECTION PROGRESS.

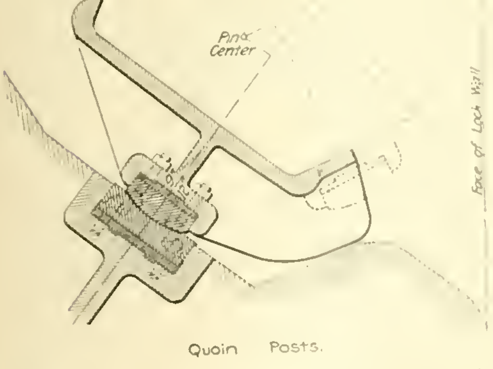
Over half of the material in the lock gates was erected during the 12 months to Sept. 1, 1913. On Sept. 1, 1912, the total quantity erected at all locks was 24,787 tons, or 42 per cent of the required 58,000 tons. On Sept. 20, 1913, the gates of the west flight at Gatun, and the east flight at Pedro Miguel, were ready for the passage of vessels, and the west flight at Miraflores locks was similarly completed by the end of the month. All gates at Gatun and Pedro Miguel were mechanically completed by Jan. 1, 1913, and those at Miraflores will be completed by March 1, 1914.



Miter Posts.

Fig. 27. Miter Bearings for Mitering Lock Gates.

Gate work at the locks began on March 21, 1911, in preparing the bearing for the upper guard gates at Gatun. The erection of these leaves began on May 17, 1911, and they were the first to be swung closed. This occurred on Aug. 4, 1912, and the footwalk over them was completed in September, 1912. In September, 1912, all leaves in the upper and intermediate levels at Gatun had been erected to full height, and work had begun on the safety gates in the lower level and the lower guard gates. The first girders for these eight leaves were placed on Aug. 29, 30 and 31, 1912; the lower guard gates were closed, and sea water admitted against them, on June 11, 1913. On Sept. 1, 1913, erection, reaming, and riveting for the gates of Gatun locks were all within less than one-half of one per cent of completion.



Quoin Posts.

Fig. 28. Quoin Bearings for Mitering Lock Gates.

At Pedro Miguel, gate work began at the upper guard gates, on Aug. 7, 1911. On Sept 1, 1912, these gates had just been closed, the skeletons of all leaves were at full height, and those in the upper level had been sheathed. On Sept. 1, 1913, erection and reaming for all gates was within 1 per cent of completion, and the finishing work was over 80 per cent completed.

At Miraflores locks, work on the gates began first on the upper guard gates and the safety gates of the upper level, in August, 1912. The upper guard gates were closed about the middle of June, 1913, and the lower guard gates on Aug. 20, 1913. On Sept. 1, 1913, all 28 leaves had been erected to full height, reaming was 98.5 per cent completed, riveting 90 per cent, and finishing about 65 per cent completed.

The local force of the contractor for the fabrication and erection of the lock gates, was 124 men in May, 1911, at the beginning of erection. It attained a maximum of about 5,300 men, 625 of whom were gold employes, during the months of March, April and May, 1913. The force on Sept. 22, 1913, was 1,940 men, 225 of whom were gold employes.

Emergency Dams.

Twin bridge dams are located above each set of locks as indicated by Fig. 18. These dams are designed to close quickly the 110-ft. channel approaches to the lock chambers in case of accident to the lock gates. All six bridge dams are alike and only one need be described.

The bridge dam consists of two vertical continuous trusses with unequal arms, pivoted on the bank of the channel and capable of being turned from a position parallel to the channel to a position in which the long arm spans the channel. This long arm carries the booms, horizontal truss, wicket girders and rolling gates which form the dam proper, as indicated further on. The pivoting, swing bearing, short arm counterbalance, etc., of the "bridge" resemble those of ordinary swing bridges. Referring to the section Fig. 31, it is observed that the floor between trusses is located above the bottom chords. Extending outside of the trusses at the floor level are boom trusses *A. A.*, with their outer edges suspended from the top chords of the "bridge." From the downstream boom is hung a horizontal truss *B*, which takes the stresses due to head of water and transmits them to abutments on the channel banks. Near the inside chord of the horizontal truss are hinged wicket girders spaced 9 ft. 2 ins. on centers. When the dam is in service these wicket girders are "lowered" with their free ends bearing against a sill on the channel between Normally the wicket girders are raised to a horizontal positon with their free ends attached to the hangers *C*.

To span the intervals between wicket girders shutters or gates are employed. These shutters are shown in position in Fig. 31. Normally they hang suspended from the floor beams. Each shutter is long enough to span the bay between two wicket girders and to project half a bay on each side. They have rollers which ride on the wicket girders and keep them to line and direction; they are placed by rolling them down the wicket girders. The lowermost row of shutters is first lowered entirely across the channel, then the second row, and so on until all five rows are placed. At mid bay where the shutters of each row come end to end there is a 1-in space between ends of shutters. To close this a 6-in iron pipe is lowered endwise down the face of the shutters and rolled along laterally until it drops into the 1-in space and closes it

The general arrangement for operating the wicket girders and shutters is indicated by the section *CD*, Fig 31 A total operating force of 16 men is required to operate a bridge dam, and a total of 218 HP. is required for the operation. The following is the recorded progress to Sept. 1, 1913, of the emergency dam erection work:

The placing of steel in the emergency dams began in August, 1912, with the laying of the circular tr k and pivoted base for the east
dam at Gatun. Erection of superstructu
proceeded directly thereafter and the da
was completed, except for final painting,
May, 1913. The contractor began the fin
operating tests on May 20 under supervisio
of testing engineers of the Canal Commi
sion Erection of the west dam at Gatu
began about the middle of November, 191
and tests were completed on Aug. 15, 191
This dam was swung across the chamber, ar

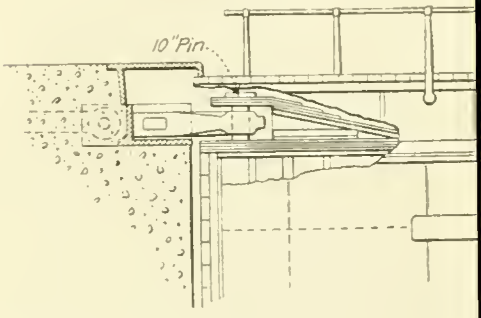


Fig. 29. Top Hinge for Leaf of Miterin
Lock Gates.

its wickets and gates dropped into place,
35 minutes.

At Pedro Miguel, erection of the west da
was undertaken first, beginning on Feb.
1913. This structure was first swung acro
the approach chamber on Sept. 2. Erecti
of the east dam, which began on April 1, w
about 98 per cent completed on Sept. 1.

The Miraflores east dam erection began
June 1, and that for the west dam on Ju
1. All material for all of the dams, weig
ing a total of 13,100 tons, had arrived on t
Isthmus by Aug. 1, and erection was 80 p
cent completed on Sept. 1, being practica
confined to the two dams at Miraflores. The
are slightly lighter than those at Gatun a
Pedro Miguel, because of the lower head
water against which they will be used; ea
of them weigh approximately 1,987 tons,
compared with 2,305 tons at Pedro Migu
and Gatun locks. The east dam at Miraflor
was to be finished the latter part of Octob
and the west dam in November.

Gate Operating Machines.

After studying all the best known types
machinery for moving the gates, it appear
that none could be counted upon to pro
satisfactory. The gate leaves are of so gr
size that more than usual care had to be e
ercised to regulate the force applied to t
leaf in a manner approximately proportion
to the resistance to its motion The resi
ance is greatest when the leaf is near the t
extremes of its path; i. e., when near t
mitered position or the position of rest in t
recess At these times there is the least ope
ing through which the water, displaced by t
moving gate leaf, can pass around the edg

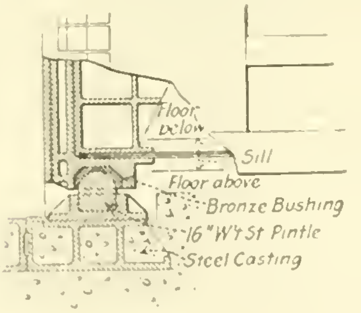


Fig. 30. Pintle or Bottom Pivot for Miter
Lock Gates.

of the leaf, and there is, therefore, the gre
est tendency for it to pile up behind the l
and form a head resisting the motion.
result of the study of the problem was, fi
to give the recess in the wall into which
leaf fits when open, such a shape as to p
mit free exit of the water around the m

post of the gate when near the position of rest; and, second, to adopt a type of machinery in which the force exerted increased and the rate of motion decreased greatly near the beginning and end of the movement.

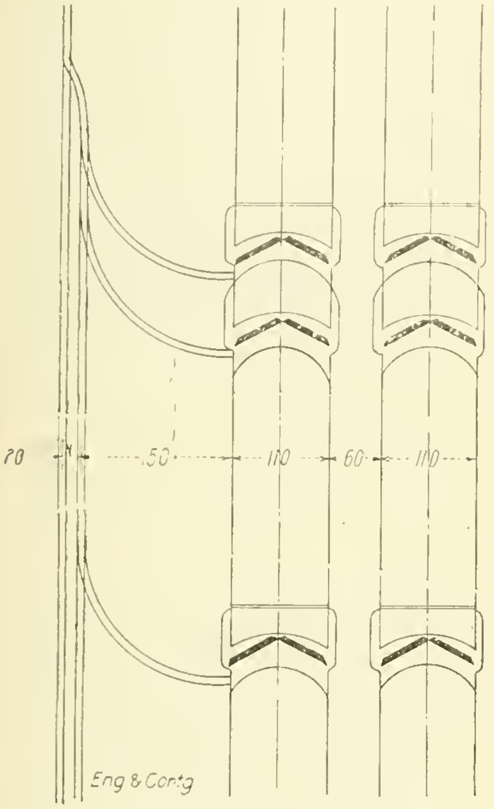


Fig. 31. Service Track Arrangement for Lock Gate Erection.

The motion is imparted to the leaf by a horizontal strut connected by a vertical pin to the upper girder of the gate leaf. The other end of the rigid strut is fitted to a crank pin attached to a large horizontal gear wheel near its circumference. The gear wheel is caused to turn by a pinion or pinions, revolving on a vertical axis, and actuated by a motor through a suitable train of reducing gear. As the large gear wheel is turned the effect upon the strut is practically that of a crank upon a connecting rod. The force in the direction of the strut approaches infinity at the beginning and end of the stroke, at which time the motion in the same direction approaches zero. The rate of travel of the gate increases gradually from the beginning to a point just beyond the middle of the path between the recess and the miter. After passing its maximum, the rate gradually diminishes until, just at mitering, it becomes very small.

The machine is capable of exerting its greatest force on the strut at the time when the resistance is the greatest. The rate of increase of the force, however, from the minimum to maximum, is much greater than the rate of increase of the resistance. It follows, therefore, that less torque will be required of the motor as the gates move from the recess than later when the gate has attained its highest speed. The horizontal slit in the face of the lock wall in which the strut moves is placed with its lowest point about 6 ins. above the highest water level in the lock. The chamber in which the largest gear wheel revolves will therefore not be actually flooded, except from some accidental cause. It is nevertheless liable to be kept continually wet by the action of the waves occasioned by vessels entering or leaving the lock and by gates in opening and closing. For this reason the chamber in which the motor operates is separated from the chamber containing the gear by a water-tight diaphragm, the motor shaft passing through a stuffing box in this diaphragm. The effect of the water on the main gear wheel and pinions will not be injurious to any great extent. There are many cases in which similar mechanism for operating the gates and valves of locks is kept continuously under water. Drainage is provided to get rid of the small amount of water entering the chamber. There are 92 of these machines, one for each gate leaf.

Valves.

The valves which control the admission of water to the filling culverts are of two types; Stoney gate valves and cylindrical valves. Designs for these gates and valves were prepared and two sets of each were ordered and tested to prove their efficiency before contracts were made for all.

Stoney Gates.—The Stoney gate valves were similar to those used at the Gatun and Miraflores spillways; each consisted of a rectangular gate of steel frame and plates seated against a chain of live rollers and provided with water seals at bearings. As these gates had in some cases to operate against a head of 60 ft., they were made exceptionally strong of nickel steel and steel castings. Each gate operates in a rectangular slot which intersects the culvert; this gate slot extends above the culvert into the lock wall far enough to permit the gate to rise clear of the culvert opening. At its top the gate slot widens into a machinery chamber and the top of the slot is sealed from the chamber by a watertight bulkhead. The rising stem from the gate extends up through a stuffing box in the bulkhead and connects with a cross head whose ends have rollers riding on vertical guides. This crosshead is actuated by two vertical non-rising screws, driven by an electric motor. The crosshead also actuates the trains of live rollers by means of vertical stems passing through the watertight bulkhead. This stem is connected to the crosshead by chain which runs through sheaves so arranged that the stem arises at half the space of the crosshead and the gate. The drawings, Fig. 35, show the gate operating machinery more fully. There are 116 of these operating sets.

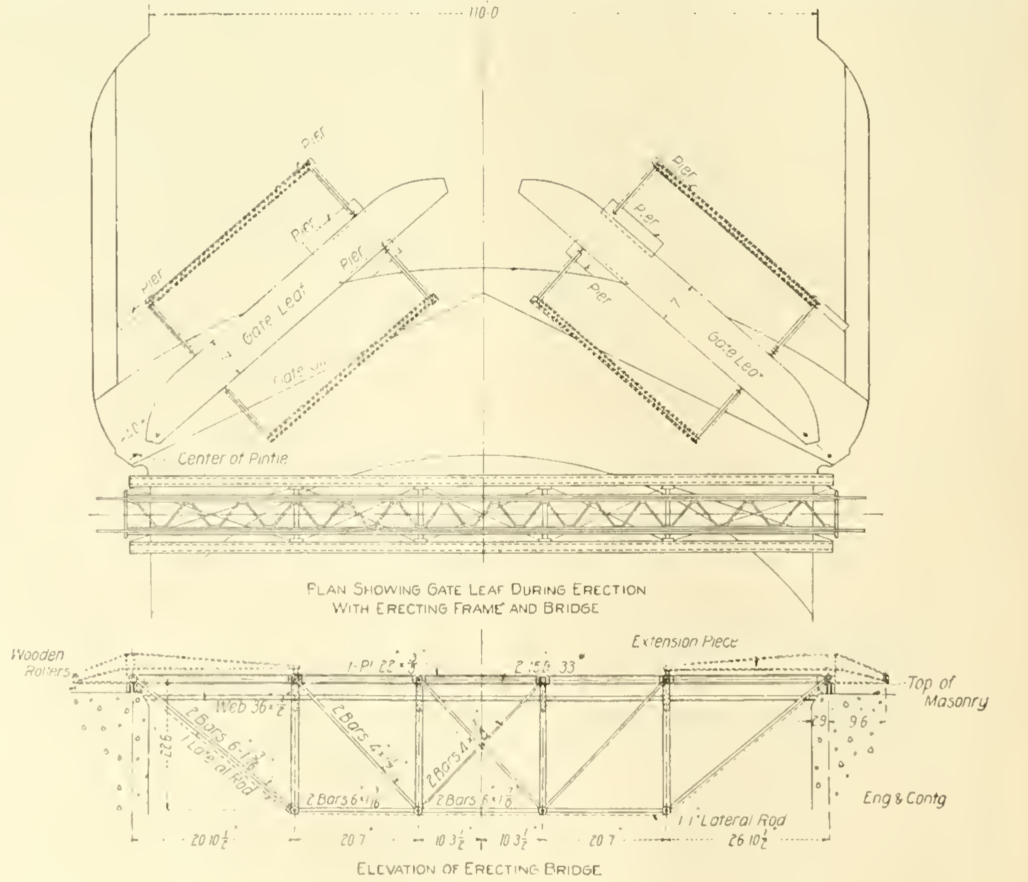


Fig. 32. Service Bridge and Erecting Frame Arrangement for Lock Gate Erection.

Cylindrical Valves.—The laterals from the center wall culvert are controlled by cylindrical valves. Details of one of these valves are shown by Fig. 36. Each valve consists of a fixed upper cylinder and a movable lower cylinder. The movable lower cylinder has a rising stem by which it is moved up or down into or out of the fixed cylinder. When the movable cylinder is down the valve is closed.

The movable drum of the valve is connected by its cylindrical stem, rising vertically through a shaft in the masonry to the machinery placed in a recess the bottom of which is 8 ft. below the coping level. The upper end of the valve stem terminates in a tubular extension carrying a stationary nut and passing through a stuffing box, closing the upper end of the vertical shaft through which the stem passes. This nut engages a non-rising revolving screw, carried in a collar with suspension bearing. The screw is turned through suitable gearing by a motor provided with solenoid brake for bringing machinery to rest when the current is off. The extension to the valve stem is prevented from turning by guides in the bedplate through which it passes. The travel of the valve in either direction is stopped at the proper point by means of a limit switch. The maximum force on the suspension bearing is exerted at the breaking of the seal when the valve is first opened. No pressure is required in closing the valve, and in case of overtravel in the downward direction the screw will rise out of the nut and slide through the thrust collar. There are 120 of these valves for the six twin locks.

Chain Fenders.

As stated above, it was determined to place at certain points in the lock flights guards for the gates in the form of chain fenders. These fenders are placed about 500 ft. above and 230 ft. below the upper and lower guard gates, respectively, in the locks at Gatun, Pedro Miguel, and Miraflores, and also at points 80 ft. and 100 ft., respectively, above the hollow quoins of the middle and safety gates in the Pedro Miguel lock and in the upper chambers at Gatun and Miraflores.

The chain fender consists essentially of a heavy chain, which, when in use, is stretched across the lock near the surface of the water, being lowered to the bottom of the chamber when a vessel is about to pass the gates it protects. Its purpose is to form a barrier to the passage of a ship, gradually stopping the latter with as little injury as possible to the vessel or the fender. To accomplish this result the chain is arranged so as to pay out against a resistance when struck by a vessel.

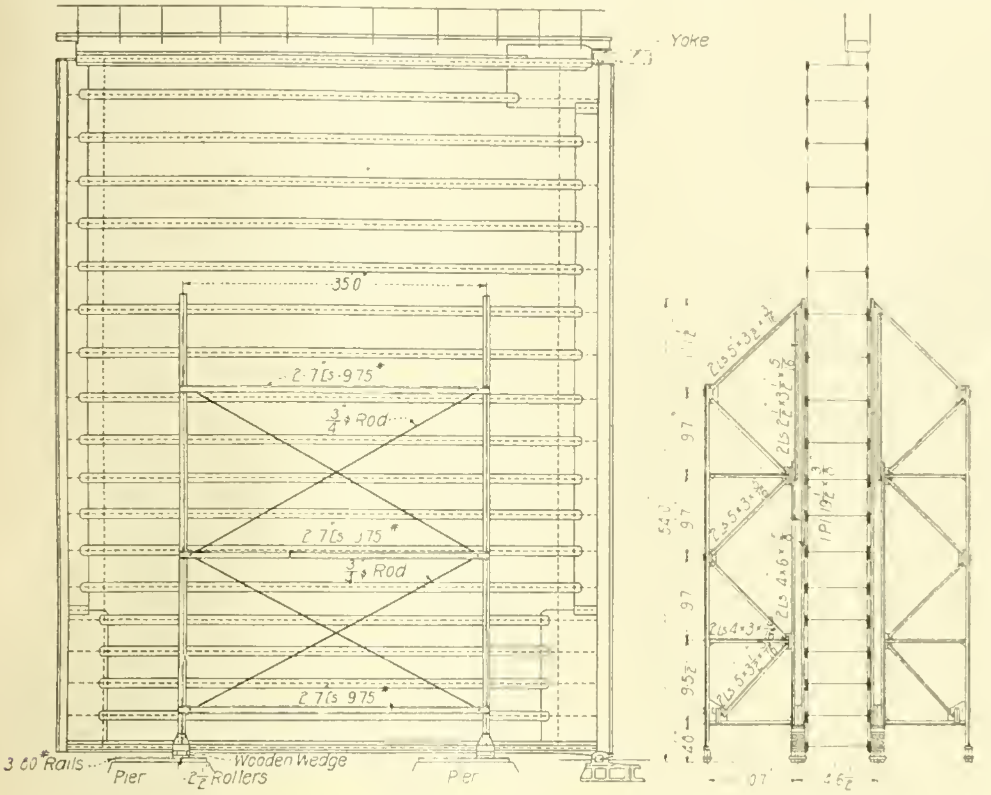


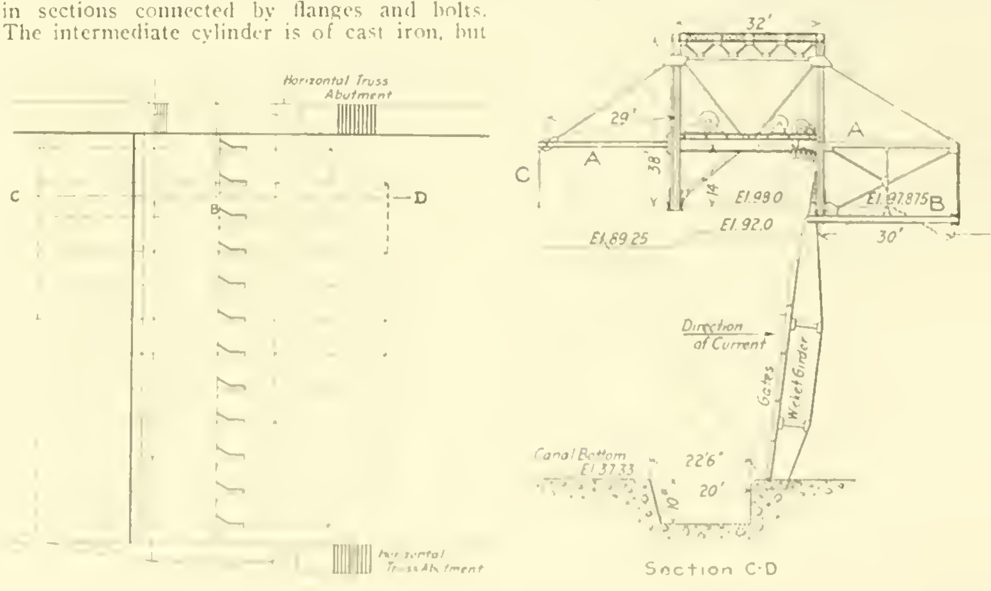
Fig. 33. Erecting Frame Supporting Gate Leaf During Erection.

The mechanism for raising and lowering the chain consists in the main of a system of hydraulic cylinders operated by a centrifugal pump, connected to an electric motor and the necessary sheaves, bearings and connections for attaching and supporting the chain. There is, at the top, a stationary outer cylinder bored out at its lower end to receive a moving combination plunger-cylinder or intermediate cylinder. This, in turn, has its lower end bored out for a stationary hollow plunger which rests on the bottom of the pit. The intermediate cylinder carries two sheaves by means of vertical eyebars attached at their lower extremities to a crosshead. Two similar sheaves are arranged at the top in stationary bearings supported on riveted girders spanning the pit. These sheaves are at right angles to the lower sheaves. The chain passes through a hawse-pipe casting of steel in the lock wall, being supported on an idler close to the edge of the wall. It makes a quarter turn around one of the stationary sheaves, then passes down and makes a half turn around one of the lower movable sheaves, rises up again, making a half turn around the second stationary sheave and going down on the other side of the machine. It then makes half a turn around the second movable sheave and passes up to one of the beams at the top of the pit, to which the end of the chain is securely fastened. The hawse pipe casting, which resists the entire horizontal pull on the chain, is secured to heavy riveted steel anchors deeply embedded in the concrete.

The upper end of the outer cylinder is attached to the girders which extend across the top of the pit, to the upper side of which the stationary bearings for the upper sheaves are bolted. The large vertical reactions of the outer cylinder and the upper sheaves are therefore on the structural frame. The tension in the horizontal portion of the chain, leading from the first stationary sheaves to the hawse pipe, is balanced by a riveted strut connecting the horizontal beams to the hawse pipe casting. The unbalanced moment due to this arrangement is small and is readily taken care of by the structural girders embedded in the concrete.

The lower end of the stationary plunger is attached to the bottom of the pit by means of a base casting, securely fastened by anchor bolts. The intermediate cylinder is moved vertically by introducing the water through the pipe leading to the top of the outer cylinder for a downward movement which raises the chain, or through the pipe leading to the bottom of the stationary plunger for an upward movement which lowers the chain. The maximum pressure required for this movement is about 60 lbs. per square inch.

The large stationary cylinder at the top has an internal diameter of 40 ins. and the metal is 2 ins. thick. It is built of cast steel in sections connected by flanges and bolts. The intermediate cylinder is of cast iron, but is provided with horizontal ribs at close intervals to withstand the heavy external pressure to which it may be subjected. The cover is to be of cast steel. The thickness of the metal in this cylinder is about 2½ ins. The joints between the intermediate and the outer cylinder and also between the intermediate cylinder and the stationary plunger are kept tight by stuffing boxes and glands, which ma be tightened without removing a single part o interfering with the working of the machin Any leakage which may occur at these stuffin boxes can be detected on the outside of th machine. The system of plungers, being o necessity rather heavy, has the further ad vantage of counterbalancing in part the ten sion of the chain when being raised, also whe it is finally stretched across the top of th lock chamber.



Section C·D

Fig. 34. Diagram Plan and Section of Emergency Bridge Dams at Locks.

The sizes of the cylinders have been base on the stress in the chain and the permissibl size of the pits in the concrete walls. Th emergency tention in the chain was fixed a 220,000 lbs. or nearly 100 gross tons, whic is about 40 per cent of its breaking loa The depth of the lock chamber is in som cases as great as 81 ft. The permissible dept of the pit in the wall is about 55 ft. Th arrangement of sheaves gives a four-fold re duction, the travel of the intermediate cylinde being about 21 ft. This will allow the chai to pay out 85 ft. from each lock wall, which i sufficient for lowering it to the grooves i the floor of the lock chamber, and is als sufficient to allow an ample travel of th chain when stopping vessels.

In addition to the cylinders, sheaves, etc. described above, there are required for th operation of the chain, that is, for lowerin and raising it into position, a centrifuga pump and operating valve, two check valves the necessary piping, and an open water tan

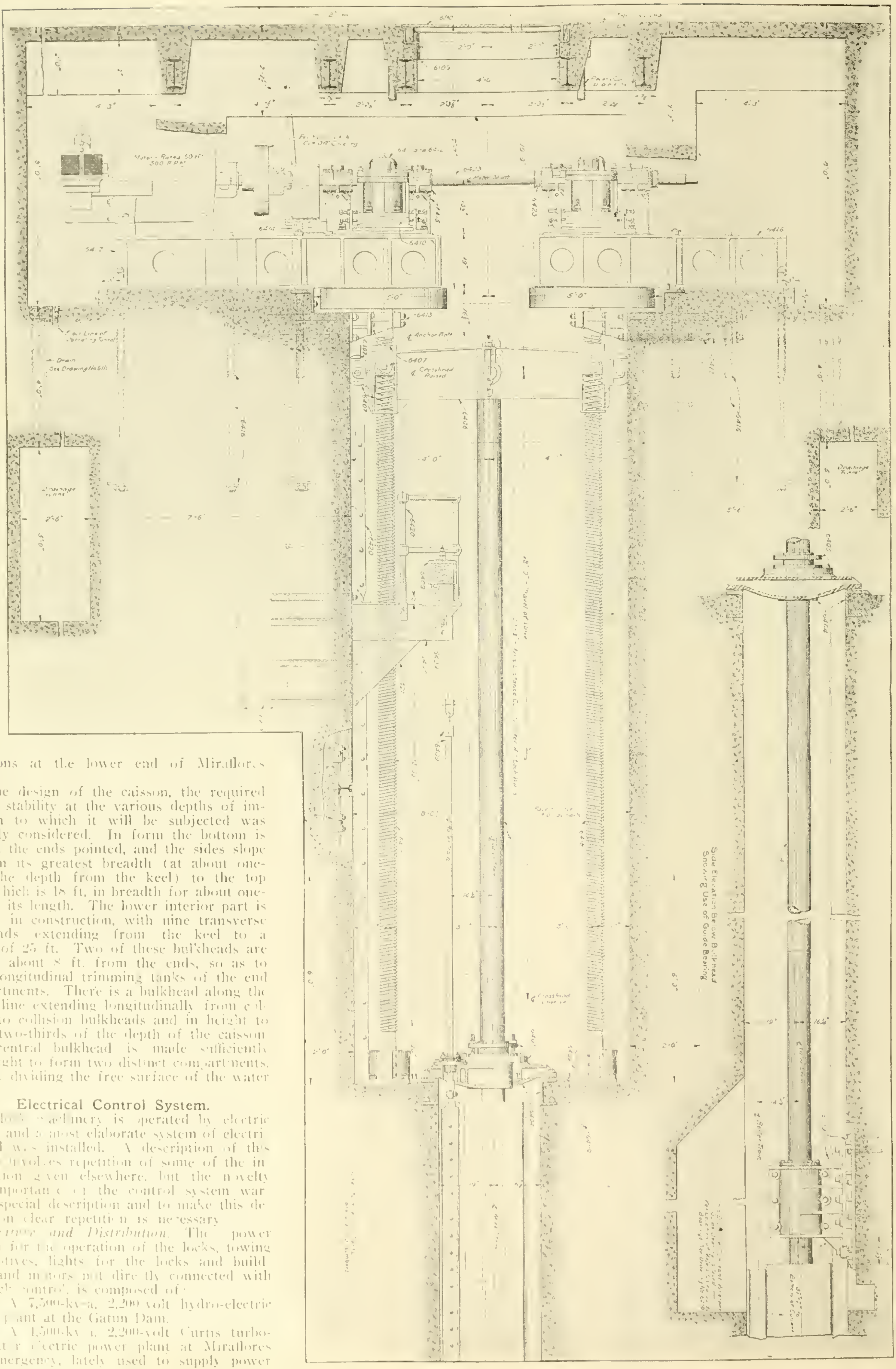
The resistance to the paying out of th chain, which is, of course, the most importan feature of the design, is provided by one o more resistance valves placed in the pip line. When the chain is struck by a ship th water pressure in the larger outer cylinde rises gradually, the resistance valve being s designed that when a pressure of about 75 lbs. per square inch in reached the valve wi open and allow a sufficient amount of wate to escape to keep this pressure constant unt the vessel is stopped. This pressure of 75 lbs corresponds to a stress of 100 gross ton on the [illegible]

Floating Caissons.

To close the head and tail bays of the lock floating caissons capable of passage throug the locks and available for use at either en are employed. Each caisson is equipped wit three 20-in. and one 8-in. centrifugal pump driven by electric motors. These are intende not only for pumping out the caisson itsel but also for unwatering the locks. The prin cipal dimensions of the caissons are:

	Ft.
Length between vertical ends	112½
Depth at side	65
Breadth molded	36
Draft, light	32
Draft, extreme	61

The great depth is made necessary by tida



conditions at the lower end of Miraflores lock.

In the design of the caisson, the required statical stability at the various depths of immersion to which it will be subjected was carefully considered. In form the bottom is convex, the ends pointed, and the sides slope in from its greatest breadth (at about one-third the depth from the keel) to the top deck, which is 18 ft. in breadth for about one-half of its length. The lower interior part is cellular in construction, with nine transverse bulkheads extending from the keel to a height of 25 ft. Two of these bulkheads are located about 8 ft. from the ends, so as to form longitudinal trimming tanks of the end compartments. There is a bulkhead along the center line extending longitudinally from collision to collision bulkheads and in height to about two-thirds of the depth of the caisson. This central bulkhead is made sufficiently watertight to form two distinct compartments, thereby dividing the free surface of the water ballast.

Electrical Control System.

All lock machinery is operated by electric power, and a most elaborate system of electric control was installed. A description of this system involves repetition of some of the information given elsewhere, but the novelty and importance of the control system warrants special description and to make this description clear repetition is necessary.

Armature and Distribution. The power system for the operation of the locks, towing locomotives, lights for the locks and buildings, and motors not directly connected with the lock control, is composed of:

(1) A 7,500-kv-a, 2,200 volt hydro-electric power plant at the Gatun Dam.

(2) A 1,500-kv-a, 2,200-volt Curtis turbo-generator electric power plant at Miraflores for emergency, lately used to supply power for construction work.

(3) A double 44,000-volt transmission line across the Isthmus, connecting Cristobal and Balboa with the two power plants.

(4) Four 44,000-2,200-volt substations, stepping down at Cristobal and Balboa, and up or down at Gatun and Miraflores, depending on which of the two plants is supplying power.

(5) Thirty-six 2,200-240-volt transmission stations for power, traction and light at Gatun, Pedro Miguel and Miraflores locks.

(6) Three 2,200-220-110 volt transformer stations for the control boards at the locks:

(7) Stations at Cristobal and Balboa for coal handling plants, machine shops and dry docks.

Power Supply and Control Panels for Lock Machinery Motors.—Current for the lock machinery and towing locomotives is transformed from the 2,200-volt system in the immediate vicinity of where it is used. There are a total of 36 transformer stations, for all locks, each containing duplicate 200-kv-a. 3-phase 2,200-240-volt transformers for power and one single-phase 25-kv-a. 2,200-220-110-volt transformer for lighting. The stations, normally fed from the 2,200-volt buses in the 44,000-2,200-volt substations, can also be operated from the power plants; the stations at Gatun locks from the Gatun hydro-electric station; and the stations at Miraflores and Pedro Miguel from the Miraflores emergency steam plant. Table XXIII gives an idea of the number and sizes of motors to be controlled in operating the lock machinery.

TABLE XXIII.

Machines and operation—	Motors each machine and HP.	Number of motors— Gatun.	Pedro M.	Mirafl's.	Total.	Tot. HP.
Miter gate, moving, each leaf	1—25	40	24	28	92	2,300
Miter gate, miter forcing	1—7	20	12	14	46	322
Fender chain, main pump	1—70	16	16	16	48	3,360
Fender chain, operating valve	1—½	16	16	16	48	24
Rising stem gate valve	1—40	56	24	36	116	4,640
Cylindrical valve	1—7	60	20	40	120	840
Guard valve	1—25	6	6	6	18	450
Auxiliary culvert valve	1—7	4	4	4	12	84
Totals	.....	218	122	160	500	12,020

There are many motors not included above, as, for instance, those for the spillway gates, for the hand rails on the mitering gates and for the sump pumps. The spillway gates are remote controlled from a special control board, and the control of the hand rail motors is given in connection with the discussion of mitering gates.

The motors are started and controlled by contactor panels located near them, the contactors of which handle the main motor currents. These contactors are controlled from the central control house. The smaller motors, including those for cylindrical valves, auxiliary culvert valves and miter forcing, are started by being thrown directly on the line. Two double-pole contactors are used, one for forward and one for reverse. In the case of larger motors for miter gate moving, rising stem valves and guard valves, a starting point with resistance in two legs of the three-phase circuit is provided.

In all cases the contactors are operated from the control boards—to be described later—by three wires, one for forward, one for reverse and a common return. In the case of panels having a starting point, the period during which the motor remains on the resistance is automatically controlled by a dashpot, so that the starting operation at the control house is the same, simply energizing a forward or reverse wire as the case may be. The control connections are arranged in such manner that each individual machine may be controlled locally. This arrangement provides for emergency operation should the control circuits from the central control house be out of order.

Location and Operation of Lock Machinery.—From an operating standpoint the machinery was placed below the coping of the lock walls, thus affording a clear space for maneuvering ships and protecting the apparatus from the weather without erecting numerous houses.

The mitering gates consist of two leaves pivoted on the lock walls which operate independently of each other. When the mitering gates are closed they are what might be termed clamped in this position by a device called a miter forcing machine. On the top of all mitering gates a foot walk with hand rails is provided. When the gates are opened and in the recesses provided for them in the lock walls, these hand rails would interfere with the passing of the towing locomotives, except in the case of the lower guard gates. The hand rails are therefore made to be raised and lowered. This is done by a motor under the foot walk, controlled from the lock wall. Near the approach to each foot walk a controller is located in the lock wall flush with the surface, this controller being operated by a foot push. If the gates are closed and the hand rails are down, and it is desired to cross on the gates, the foot push is pressed and the hand rails are raised by their motors. This is true not only of the hand rails on the nearer gate leaf, but of the hand rails on the farther leaf as well. After passing across, one can, if one desires, press the foot push on the other side and both hand rails will be lowered. Or, if one leaves the hand rails up and the gates are opened by the operator in the control house, they will be automatically lowered so as to be out of the way when the gate is in the recess. When the gates are again closed, the hand rails will automatically rise again if the foot controller has been operated in the meantime. The hand rails cannot be raised when the gates are opened, and no harm results if the foot switch is operated while the gates are in the closed position.

The chain fenders are stretched across the canal in front of all mitering gates which can be exposed to the upper lock level and also in front of the guard gates at the lower end.

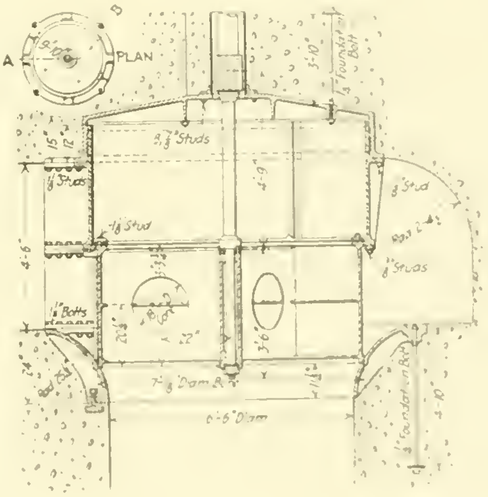


Fig. 36. Details of Cylindrical Valves for Lateral Culverts.

These chains are maintained in a taut position when the gates behind are closed, and are lowered when the gates are opened for the passage of a ship. The chains are raised and lowered by a method similar to that followed in hydraulic elevators. Lowering the chain for the passage of a vessel and raising it again after the vessel has passed is accomplished by two motors, one driving the main pump supplying water under pressure, and the other operating a valve which controls the direction of movement of the chain. These two operations are combined in one, each motor being stopped automatically by a limit switch when the motor has performed its function.

The filling and emptying of the locks is accomplished by three culverts, one in the middle wall and one in each side wall, the flow of water being controlled by rising stem valves. They are located in the culverts at points opposite each end of each lock so that the culvert can be shut off at any desired point for filling a lock with water from above or upstream, or for emptying it by allowing it to flow out and down to the next lock. Lateral culverts conduct the water from the main culverts, under the lock chambers, and up through openings in the lock floors.

The rising stem valves are installed in pairs and each pair is in duplicate; also each culvert is divided into two parallel halves a these valves by a vertical wall. This arrangement reduces the size of each valve and makes it more easily operated, each valve being 8x18 ft. One pair of duplicates is left open as a guard, or reserve pair; the other pair is used for operating, so that in case of an obstruction in the culvert or accident to the machinery, the duplicate pair can be used.

At the upper ends of the culverts at the side walls, the duplication is accomplished by three valves in parallel, called the guard valves. They perform service exactly similar to the rising stem valves, except that three valves in parallel in this case must conform to the same laws as the two in parallel in the othe case.

The culvert in the middle wall must serve the locks on both sides, and to control thi feature cylindrical valves are placed in the lateral culverts that branch out on each side. There are ten of these on each side of the culvert at each lock.

At the upper end of each set of locks, ther are two valves in the side wall for regulatin the height of water between the upper gat and upper guard gate, as it is desired to maintain the level of the water between these gate at an elevation intermediate between that o the lake above and that of the upper loc when the upper lock is not at the same leve as the lake. These valves are called th auxiliary culvert valves.

Reasons for Using the Control Systen Adopted.—As the flight of locks at Gatun, fo instance, extends over approximately 6,20 ft., and the principal operating machines ar distributed over a distance of about 4,000 ft it can be readily seen that central mechanica transmission of control of machines would b almost impossible; and to control the machines locally would mean a large operatin force distributed practically along the fu length of the locks, which has invariably bee the practice heretofore. Such a force woul be difficult to co-ordinate into an efficient operating system. The situation therefore resolved itself into centralized electrical control, which reduces the number of operator operating expense, and liability to acciden To accomplish this system of control, a control board for each lock was constructed whic permitted having all control switches locate thereon mechanically interlocked so as t minimize, if not entirely prevent, the error of human manipulations.

Centralized Control and Indicating Systen—The control boards are installed in contro houses located on the middle walls at point which afford the best view of the locks, although this view is not depended on to know the position of the gates or other apparatu as all are provided with indicators on the control board. The control boards are mad approximately operating miniatures of the locks themselves, and are arranged with indicating devices which will always show the position of valves, lock gates, chains an water levels in the various lock chambers; an with the exception of such machinery as need only an "open" or "closed" indication, the indications will be synchronous with the movement of the lock machinery.

For such indication, appliances with commutators, multiple contacts or ratchet mechanisms would not be suitable because o the many contacts and small pieces in their construction; and particularly because devices of this character move step by step and would not indicate all points in the movement of the main machinery, such indications being more or less approximate according to the number of steps in the indicating devices. The indicators on the Panama control boards were developed especially for this undertaking, and show accurately and synchronously every movement of the machinery to which they are connected, whether in the extremes of travel or at any intermediate point.

A complete synchronous indicator consists of a transmitter located at and operated by the machine in the lock wall, and a receiver operating an indicator at the switchboard in the control house. Both transmitter and receiver have a stationary and a rotating part. The stators have three-phase windings with leads from three corresponding equidistant points brought out and connected together, but not connected to a source of power, the stator coils being energized by induction from the rotors. The rotors are bipolar and are connected in multiple and energized from a 110-volt 25-cycle single-phase source.

The movement of the lock machinery and with it the connected transmitter rotor produces a field in the transmitter stator polarized in the direction of the rotor axis, which induces voltage in the stator coils. This voltage is transmitted by the three-phase connection above mentioned to the receiver stator coils and duplicates in them but in the reverse direction, the same conditions of polarity and voltage as present in the transmitter. The rotor of the receiver being energized by the external source in the same direction as that of the transmitter, is reacted upon by the polarized receiver stator until the magnetic axes coincide and the rotors of both transmitter and receiver are in the same relative position. Any difference in the position of the transmitter and receiver rotors causes a difference of potential between the stator windings with a consequent flow of current and resultant torque, which again moves the receiver rotor to the same relative position as that of the transmitter rotor. The receiver rotor follows closely and smoothly the movement of the transmitter rotor, and consequently imparts to the position indicator a movement identical with the movement of the lock machine, although on a scale reduced to the requirements of the control board. A brief description of the individual synchronous indicators follows.

In the case of the mitering gates, the vertical operating shaft is connected to a shaft which operates the transmitter machine. The latter shaft is threaded and carries a nut on which is mounted a rack. The rack engages a gear on the rotor shaft, and this turns the rotor as the gates operate. The mitering gate indicator comprises a pair of aluminum leaves, shaped to correspond to the plan view of the top of the gate, which travel horizontally just above the top of the board, the hinge ends being connected to shafts extending down through the surface of the board where they are geared to the receivers by means of bevel gears. When the miniature gates are completely opened, they are covered by shields to give the effect of the gates folding back into recesses in the lock walls.

For the chain fender, the position indicator transmitter is driven by the shaft which operates the limit switch that controls the stroke of the piston. The indication on the board is given by a small aluminum chain, which, like the large chain, is raised and lowered, each end operating independently, the large chain being lowered to the bottom of the lock and the small chain into a slot on the control board. The ends of the miniature chain are fastened to semaphore arms which are connected to segmental gears meshing with the driving gears on the receiver machines. As the receiver rotors turn, the chain is either lifted or lowered, the position of the large chain from the bottom of the lock being indicated by the angle of the semaphore arms.

As the rising stem valves occur in pairs, their position indicator machines occur in pairs also. The transmitter rotor is driven by a shaft and gearing similar to that described for the mitering gates. Each indicator is similar to a small elevator, a car being used to indicate the position of the valve gate. Both front and back of the shaft is fitted with opal glass marked with black lines for the ¼, ½ and ¾ positions. A small aluminum cage moves up and down in each compartment. A drum for operating the cord which raises and lowers the cage is located underneath the control board and is operated by the receiver through a suitable train of gears. To make the indications visible from points up and down the control board, the elevator shaft under each car is always illuminated and the portion above is dark.

Water Level Indicators.—The specifications covering the water level indication required an accuracy of 1/20 ft. or 1/10 of 1 per cent in actual water level. In the transmitters and receivers for the machines described previously, the rotors turn less than 180° with an inherent lag of 1½ per cent between transmitter and receiver rotors in this distance, which obviously prevents this arrangement from being employed to give the water level indication.

It was found that if the rotors were revolved ten complete revolutions, the required accuracy could be obtained; but since this arrangement makes it possible for the rotors to be in synchronism every 180°, or in twenty different positions for the entire travel, the indicators would not indicate correctly if for some reason the transmitter rotors were turned more than one-half revolution with the power off. Therefore, the required accuracy was obtained by two sets of transmitters and receivers, one set connected to a fine index in which the rotors make ten complete revolutions and the other set connected to a coarse index operating less than 180°.

The fine index is a hollow cylinder carrying a pointer, the length of the cylinder being such that when an aluminum ball representing the coarse index, which can be depended upon for coarse indication, is within the limits of the cylinder, the reading of the fine index is correct within the limits specified. The scales are illuminated by lamps in both base and top caps of the indicator.

For water level indication, wells 36 ins. square in the lock walls with communication to the lock by a small opening at the bottom of the wall to deaden surges contain a welded steel box float, 30 ins. square by 9 ins. deep. A non-slipping phosphor bronze belt transmits the movement of the float to a sheave fitted with pins on the transmitter mechanism, the pins registering with holes punched in the belt. The sheave shaft is carried in ball bearings with oil cups for lubrication and drainage cocks at the bottom of the bearings.

The position of the miter forcing machine is not indicated by synchronous indicators, but its open and closed positions are shown by red and green lights and a mechanical indicator on the control board representing the machine.

Control Boards Represent Locks in Miniature.—The control boards are of the flat top benchboard type, 32 ins. high by 54 ins. wide, built in sections, with total lengths as follows:

Gatun	64 ft.
Pedro Miguel	36 ft.
Miraflores	52 ft.

The side and center walls of the locks are represented by cast-iron plates and the water in the locks by blue Vermont marble slabs. The outer edge of the board is surrounded by a brass trim rail, and the sides are enclosed with steel plates which can be readily removed for inspection of the board. The control board is supported by a wrought iron framework resting on base castings, which are in turn supported on the operating floor of the control house.

The control switch handles are mounted above the surface of the board and operate through an angle of 90°. They are provided with nameplates for the "open," "closed" and "off" positions. The space immediately below the flat top of the control board is occupied by the contact fingers of the control switches, mounted on the operating shaft, synchronous receivers and their cable connections. Connection boards are provided for the cables, which are led up from each side, as are buses for supplying current to the control switches, receivers and the lamps that illuminate the scales of indicators. The receivers, transmitters and lamps are operated at 110 volts, while the control circuits are 220 volts, both using 25-cycle alternating current.

Mechanical Interlocking System.—In order to make it necessary for the operator to maneuver the control switch handles always in a certain order, corresponding to a predetermined sequence of operation of the lock machinery, and to prevent the operator in control of one channel from interfering with the machinery under the jurisdiction of the operator controlling the other channel, these control switches are provided with interlocks. The interlocks are in two vertical racks under each edge of the board and some distance below, so that they may be inspected and oiled from a floor which is about 7 ft. below the floor on which the switchboard operator stands. The latter floor does not extend across under the board, this space being open so that all parts on the underside of the board are accessible from the floor below.

Vertical shafts operated by connecting rods from the control switch shafts extend downward past the electrical parts for the operation of the interlocks. The interlock system is essentially a bell crank mechanism, connecting the shaft of the control switch through a movable horizontal bar to a vertical operating shaft which can or can not move according to the relative positions of the interlocking bars and dogs. The interlocking rack is a steel frame carrying five horizontal members. Upon these and tying them together are vertical steel straps which carry brass runway posts for the vertical and horizontal interlock bars. These posts are riveted to the vertical steel straps, a thin brass plate between posts and straps making the runways noncorrosive. The vertical operating shafts are of square steel turned on the ends and work in brass bearings near top and bottom of the interlocking rack. Forked cranks mounted on the vertical operating shafts move the horizontal interlock bars by means of pivot blocks set over pin blocks riveted to the horizontal bar. The interlock bars and dogs are of special shape and hard extruded brass, which section keeps the dogs in line with the axis of the bars when under pressure by being engaged with another dog on a vertical bar. Every control switch uses a horizontal bar of from 3 to 50 ft. long.

The interlock system depends mainly on the action of engaging bevel dogs located on horizontal and vertical bars, the movement of a horizontal bar tending to lift a vertical bar by bevels on the dogs. A horizontal bar can not be moved without raising a vertical bar. Thus if at any time a dog on a horizontal bar rests against the upper end of a dog on a vertical bar, no movement of the horizontal bar where the dog engages with the vertical bar can take place, and the control handle connected to that particular horizontal bar is locked.

Interlocks prevent the chain fender from being lowered until adjacent mitering gates have been opened, and also prevent the gates being opened until the chain is in the raised position. In this way it is assured that the chain fender will always be in the up position to protect the gate when the gate is closed. To avoid unnecessary complication, each end of the chain is interlocked with the leaf on its side of the lock only, because as a rule both leaves of a gate, as well as both ends of a fender chain, will be opened simultaneously, and further interlocking is unnecessary. After the mitering gates are closed, a miter forcing machine is operated by a control handle and locks the ends of the gates closed. This machine cannot be operated until the gates are closed.

Also the rising stem valves of the side wall, next above or below a miter gate, must be closed while the miter forcing machine is open. As the miter forcing machine cannot be closed until the gates are closed, this means that the valves either above or below the gate must remain closed until the gate itself is closed, thus preventing the operator from creating a current of water around the gates while they are open, or being moved in opening or closing. This interlock is not included on the middle wall valves for the reason that they will be used with the locks on either side and must be free for that purpose.

Either pair of rising stem valves may be opened first, at the choice of the operator, an interlock becoming effective when the first valve of the second pair of duplicates is opened. This is done by a novel arrangement of equalizing levers acting against the ends of the interlock bars, with certain definite amount of lost motion which is taken up on opening the first pair of valves, thus putting the interlocks in operation on the next pair. To illustrate this operation, consider, for example, a side wall culvert at Gatun with its principal rising stem valves at each change of level from one lock to the next. The control of these valves is interlocked so that if the valves are opened at one particular point, the valves a lock length upstream or downstream cannot be opened. Thus the operator is limited to equalizing the water between locks and cannot allow water to flow from the upper lock past the middle lock into the lower lock, which operation, if permitted, might flood the lower lock walls and the machinery chambers in them. The cylindrical valves are interlocked so that if those on one side are locked opened the ones on the other side are locked closed, and the opening of one switch on a side will lock the opposite ten. This prevents careless cross filling between locks, which operation might be combined with the regular method and produce flooding. However, there may be times when it is desirable to employ cross filling to economize in the use of water from Lake Gatun in the dry season. For this reason this interlock is made removable by the use of a Yale lock and key. The key will be placed in the hands of the chief operator.

In the use of the middle wall culvert, the cylindrical valves on one side or the other must be opened before the rising stem valves can be opened, and the rising stem valves must be closed first. This interlock is applied in order to require the operator to control the flow of water by means of the rising stem valves rather than by the cylindrical valves.

In most cases the locks are divided into two unequal parts by the intermediate mitering gates. This arrangement makes it necessary to divide the ten cylindrical valves into two groups of seven and three, respectively, for the long and short lengths. A selecting lever is provided for these interlocks and may be set as indicated by a nameplate on the lever to "three," "seven" or "ten," respectively; whereupon the corresponding valves are subject to that interlock, and the others of the group of ten are locked closed if three or seven only are to be used. The failure of the operator to make his selection properly in advance will simply cause him the trouble of going back and doing so, as the remaining valves are locked closed. This arrangement permits handling small vessels without causing waste of water due to operating such vessels in the large chambers. If a short vessel were being passed downstream, it would first pass into the chamber having three cylindrical valves. The group selective lever would then be placed on the "three" position which would permit the opening of three valves above the intermediate gate, but would lock closed the other seven valves above it. After the vessel had been passed below the gate the handle may be reversed releasing the lever and locking three switches.

TABLE XXIV.

	Gatun. (Per cent completed)			Pedro Miguel. (Per cent completed)			Miraflores. (Per cent completed)		
Machinery—	No.	Mechanical.	Electrical.	No.	Mechanical.	Electrical.	No.	Mechanical.	Electrical.
Cylindrical valve machines.....	60	100	42	20	100	87	40	100	38
Rising stem valve machines....	56	99	56	24	100	71	36	86	26
Miter gate moving machines....	40	93	28	24	90	58	28	83	4
Auxiliary culvert valve machines	4	100	72	4	100	62	4	100	45
Spillway gate machines.........	14	71	5	..	..	..	8	0	0
Guard valve machines..........	6	100	40	6	96	5	16	33	0
Miter forcing machines.........	20	39	46	12	33	30	14	1	7
Chain fender machines.........	16	43	5	16	35	0	16	30	0
Handrail operating mechanism.	36	70	54	20	0	16	24	0	8
Miter gate sump pumps.........	40	69	44	24	67	0	28	43	0
Chain fender sump pumps......	16	33	0	16	0	0	16	0	0
Transforming stations	16	..	67	8	..	55	12	..	38

There are intermediate rising stem valves in the side walls at each intermediate gate, but no interlocks are applied to these for the reason that they will be used in a more or less irregular manner, and no fixed laws for their operation can be made in advance. Moreover, they control the water only between different sections of the same lock, and there is not the danger from mistakes in operation which exists in the case of the other valves which control water between lock levels. The same is true of the small auxiliary culvert valves, by means of which the space between the upper guard gate and upper main gate is filled and emptied.

In case a large vessel is to be locked through, the interlocks on the intermediate gates can be made ineffective by the operation of a Yale lock which uncouples a clutch and disconnects the central switch from the operating mechanism. Turning the key removes the interlock and permits the intermediate gates to be thrown open to obtain a 1,000-ft. level and the valves operated independently of these gates.

To obviate the possibility of flooding the locks when valves are in a certain position, diagonal interlocking is introduced between the rising stem valves of the side wall and those of the middle wall a lock length away. This interlocking between valves diagonally across a lock when the cylindrical valves are open is needed to prevent the flow of water from, say, the upper lock by way of a side wall culvert to the middle lock, thence by way of the middle wall culvert to the lower lock, thus allowing an operator through carelessness to flood the lower lock walls. If the cylindrical valves of a certain lock are closed, the interlock is not needed on the rising stem valves of that lock; and since such interlock would interfere with the proper use of the valves of its twin lock on the other side of the middle wall, this interlock is automatically removed when all ten cylindrical valves are closed on the particular lock in question, and is automatically applied again if one or more of the ten cylindrical valves are opened. Furthermore, the valves of the side wall immediately at the gate which is being moved will be open to equalize water level, and diagonal interlocking will prevent the opening of the middle wall valves a lock length above or below the gate being moved. Each of the four valves of such a group has independent control, their control switches being so interlocked that either pair may be opened and left open as guard valves, the interlocks becoming effective when the operator tries to open the first valve of the second pair. In addition to these pairs of valves in parallel, each pair is duplicated at each change of level from one lock to the next.

Special Climatic Requirements.—To withstand the humid atmosphere of the Isthmus, every insulated part, such as solenoid, relay, circuit breaker and other coils, was impregnated with non-hygroscopic compounds. All small parts were made either of brass, copper, Monel metal, bronze, or of sherardized iron or steel. Mica and treated asbestos lumber were used largely in place of fiber or wood.

Operating Machinery Erection.

In addition to the gates and emergency dams, etc., there is a great amount of auxiliary machinery contained within the machinery rooms and operating galleries of the walls or in the culverts. This apparatus embraces the rising stem gate valves, which control the flow of water in the 18-ft. culverts at the bottoms of the side and center walls; the cylindrical valves, controlling the 6-ft. culverts which run laterally from the wall culverts and pass water to the lock chambers, through the floors; the auxiliary culvert valve machines, which control the flow through a short auxiliary culvert designed to maintain the water at the same elevation on both sides of the upper guard gates; the guard valves and machines, controlling the intake of water into the side wall culverts at the upper end of the locks; the miter gate moving machines, to swing the leaves of the gates; the miter forcing machines, to draw the leaves tightly together when closed, and prevent leakage; the mechanisms which cause the hand rails on top of the leaves to fold down when the leaves are swung back into their recesses in the walls; the pumps for unwatering sumps in the miter gate leaves, and in the pits of the fender chain machinery rooms; the machinery for actuating the fender chains, at all upper and lower guard gates, and at the intermediate and safety gates in Pedro Migual and upper chambers of Gatun and Miraflores locks; the tracks and conductors for the towing locomotives; and the posts for the illumination of the locks.

The condition of the principal machines necessary to the operation of the locks on September 1, 1913, is indicated in Table XXIV.

All designing work for the canal locks and appurtenances for all other canal structures was under the direction of Col. H. F. Hodges. Under Col. Hodges were the following chief assistants in charge of special features: Operating Machinery, Edw. Schildhauer, electrical and mechanical engineer; Aids to Navigation, W. F. Beyer, assistant engineer; Masonry and Lock Design, L. D. Cornish, designing engineer; Lock Gates and Protective Devices, Henry Goldmark, designing engineer; Emergency Dams, T. B. Monniche, designing engineer. The construction work of the Gatun Locks was under the direction of the engineers of the Atlantic Division and the construction work of the Pedro Miguel and Miraflores locks was under the direction of the egnineers of the Pacific Division, Mr. Sidney B. Williamson, division engineer.

Culebra Cut

Of the several structural features of the Panama Canal, the Culebra cut doubtless holds first place in public opinion. This judgment is not far in error from the viewpoint of the engineer. The date of the possible completion of the cut set from the first the date of the completion of the canal. It was known also from the first that it was the most expensive single feature of the canal work. These estimates of magnitude, morever, were based on the 53,000,000 cu. yds. estimated excavation of the Minority Report of the International Board of Consulting Engineers. In 1902 the decision to widen the channel through Culebra added over 13,000,000 cu. yds. to the theoretical excavation. Then the slides began to develop and every year saw them add several million yards to the work until finally on July 30, 1913, over 93,000,000 cu. yds. had been removed from the cut. The final amount will be still greater but probably only a few million yards more.

General Methods.

The general method of excavating Culebra Cut can be stated in a few words. The excavation was dry cut through earth and rock, about 71 per cent of the yardage requiring blasting. The problem was the same as that at any rock cut for a railway in the United States, but it was that problem magnified many times. The rock was broken up by blasting and excavated and loaded into cars by steam shovels, and the cars were hauled in trains to waste dumps or to where the spoil could be utilized in other construction. The rock was taken out in cuts parallel to the canal axis. The shovels on short tracks worked against a face or bench; the trains which received the spoil stood on tracks parallel to the benches, and these loading tracks connected with main tracks leading to the places of spoil disposal. The drills worked ahead of the shovels.

The work separates for consideration into the following items: Drilling and blasting; steam shovel operation; transportation; spoil disposal; excavation records; excavation costs; slides and methods of handling them.

Drilling and Blasting.

The general plan pursued was to drill lines of holes parallel to the benches and blast down the faces. This general plan was of course subjected to many modifications to meet conditions that developed and called for changes in detail.

Drills and Drilling.—The drills used were of two general types, well drills and percussion drills and with few exceptions they were operated by compressed air. Drilling operations prior to July 1st, 1907, were of moderate extent and the great bulk of the drilling had been completed by July 1, 1912. The record of drill work in detail will therefore be confined to the five years of major operations. It is tabulated as follows:

1907-1908.

Items—	Amounts.
241 air drills, lin. ft. hole	1,224,902
66 well drills, lin. ft. hole	613,568
Hand drilling, lin. ft. hole	210,461
Total hole drilled, lin. ft.	2,048,931
Yardage mined	8,106,716
Yards mined per foot of hole	4

1908-909.

269 tripod drills, lin. ft. hole	2,062,482
143 well drills, lin. ft. holes	1,306,970
Hand drilling, lin. ft. hole	267,243
Total holes drilled, lin. ft.	3,636,696
Yardage mined	12,622,880
Yards mined per foot of hole	3.5

1909-1910.

233 tripod drills, lin. ft. hole	2,019,176
153 well drills, lin. ft. hole	2,063,067
Hand drilling, lin. ft. hole	272,461
Total hole drilled, lin. ft.	4,354,754
Yardage mined	12,158,996
Yards mined per foot of hole	2.8

1910-1911.

227 tripod drills, lin. ft. hole	2,274,080
155 well drills, lin. ft. hole	2,217,962
Hand drilling, lin. ft. hole	308,347
Total hole drilled, lin. ft.	4,800,359
Yardage mined	11,672,241
Yards mined per foot of hole	2.4

1911-1912.

230 tripod drills, lin. ft. hole	2,368,804
150 well drills, lin. ft. hole	2,243,194
Hand drilling, lin. ft. hole	243,302
Total hole drilled, lin. ft.	4,855,300
Yardage mined	12,862,968
Yards mined per foot of hole	2.6

The total length of hole drilled during the five years was 3,730 miles, or, adding the footage drilled prior to July 1, 1908, and after June 30th, 1912, something over enough to reach to the center of the earth.

Explosives and Blastings.—The Culebra excavations required some 3,000 tons of explosives per year. Prior to June, 1908, explosives received from the manufacturers in the United States were stored at Bas Obispo and at Gold Hill. About the date named two concrete block store houses were erected, one at Mindi and one at Gamboa. Each of these stores comprised a 300-ton dynamite magazine, a detonator house and a watchman's house. With the installation of these stores a system of explosives service was established as follows: One month's supply plus 25 per cent was maintained in each magazine and one month's supply was kept in transit from the United States. The bulk of the explosive was 60 per cent and 45 per cent dynamite but several patented explosives and also powder were used in smaller amounts. The quantities of explosives used during the five years, July 1, 1907, to July 1, 1912, were as follows:

Year—	Tons.	Lbs. per cu. yd. mined.
1907-08	2,352	0.64
1908-09	3,365	0.59
1909-10	3,157	0.49
1910-11	2,638	0.50
1911-12	2,603	0.45

The general method of blasting was to fire a series of vertical holes parallel to the face of the bench and a series of toe holes drilled horizontally at or nearly at the foot of the bench. Until June 30, 1908, all holes were fired by blasting batteries of the usual magneto-electric type, but so many mis-fires resulted that an electric firing current wire was strung the entire length of the cut. To this wire 45 blasting spurs about 1,000 ft. apart were connected. Each of these spurs terminated in a 5-kw. 110-volt transformer with a switch box and secondary leads. From the switch box No. 4 triple insulated wire was laid on the ground in the cut, and all blasting operations were performed by wiring fuses in parallel and connecting them to the leads of No. 4 wire. The current used was a 60-cycle, 2,300-volt current which the transformers stepped down to 110 volts. This arrangement practically eliminated miss-fires. Blasting batteries were subsequently used for springing holes, or for firing isolated groups of a few holes. As a further precaution against miss-fires due to defective caps, all powder men were supplied with galvanometers and required to test caps before and wiring after the holes were connected up for blasting. These precautions reduced accidents to a very few. During the three years, June 30, 1909, to June 30, 1912, with over 18,800,000 lbs. of explosive fired only five men were killed by explosions.

Summary.—The records made public permit no very detailed analysis of the drilling and blasting at Culebra but the following data reported at various times give some indication of performance. The drilling and blasting costs based on yardage actually mined and not on yardage excavated were as follows:

Year—	Per cu. yd.
1908-09	17.36 cts.
1909-10	17.27 cts.
1910-11	16.49 cts.
1911-12	15.23 cts.

From data covering the twelve months to July 1, 1912, it is found that the average number of feet drilled per tripod drill per day was 41 ft., at an average operating labor cost of 8.96 cts. per linear foot. For well drills the daily average was 50 ft. of hole drilled at an average operating labor cost of 6.68 cts. per linear foot. The average depth of vertical hole was 19 ft. and the average charge per hole was 24 lbs. of explosives; the average length of toe hole drilled was 15 ft. and the average charge per hole was 30 lbs. An average of 2.21 cu. yds. of material was broken up per pound of explosive used. These included "dobe" shots or "mud caps." There were 31,227 "dobe" shots fired during the year at a charge per shot of 6.76 lbs. of explosive and blasting material. Detonators and safety fuse were used for firing. The cost per "dobe" shot averaged 90 cts.

Steam Shovels.

Steam Shovel Operation.—All blasted rock at Culebra was excavated and loaded onto trains by steam shovels. These machines were used in varying numbers and of different sizes and to present their performance it is necessary to subdivide it into the years of record. The five years from June 30, 1907, to June 30, 1912, having included the great bulk of all steam shovel work are alone considered.

Plant and Performance.—Five sizes of shovels were employed as follows:

Name—	Dipper capacity.
45-ton Bucyrus	1¾ cu. yds.
70-ton Bucyrus	2½ cu. yds.
95-ton Bucyrus	5 cu. yds.
No. 60 Marion	2½ cu. yds.
No. 91 Marion	5 cu. yds.

The number of each type used each year varied; the number of all shovels used each year and the average output of all shovels are given in Table XXV.

TABLE XXV.

Year—	1907-08.	1908-09.	1909-10.	1910-11.	1911-12.
Shovels worked	59	75	61	52	46
Number working days	305	304	304	304	304
Cu. yds. per day	932	1,199	1,231	1,314	1,319
Cu. yds. per month	23,685	30,371	31,185	32,635	33,038
Cu. yds. per hour at work	199	225	236	241	246
Cu. yds. per hour under steam	121	150	156	166	165

These performance records and all others which will be stated are based on an eight-hour working day. The best performances per day, month and year of steam shovels of the sizes named above for the five years are given in Table XXVI.

TABLE XXVI.

Item—	45-ton Bucyrus.	70-ton Bucyrus.	95-ton Bucyrus.	No. 60 Marion.	No. 91 Marion.
High daily, date	Feb. 5, '08	Mar. 19, '12	Mar. 22, '10	Apr. 18, '08	June 21, '09
High daily, yardage	1,356	2,900	4,465	1,904	3,485
High monthly, month	July, '08	Mar., '09	Mar., '10	Mar., '08	Aug., '08
High monthly, yardage	25,713	53,043	70,290	41,219	55,419
High monthly, days worked	26	27	26	26	25
High annual yardage	105,740	300,872	543,481	....	441,927
High annual days worked	131	254	295	....	299

Maintenance of Shovels.—Steam shovels were maintained by shop and field repairs. Prior to Oct. 1, 1909, all shop repairs of steam shovels were made at the Empire shops and were in charge of the Mechanical Division and all field repairs were made by the construction division. On the date named the Empire Shops were removed from the direction of the Mechanical Division and transferred to the Central Division, and were made virtually steam shovel repair shops for the whole canal.

A field repair system was begun Nov. 5, 1907, and it comprised a force of boiler workers, pipefitters, machinists and helpers who made all repairs at night after the shovels were shut down. The force was divided into three gangs, each covering a certain section of the excavation. A machine shop car, equipped with a forge, drill and shaper and carrying necessary small tools, and a locomotive crane constituted the field repair plant. The field repairs included replacing and repairing circles, booms, dippers and dipper sticks, A-frames, hoisting drums, main and propeller shafts, swinging drums, intermediate shafts, water tanks, feed pumps and trucks and in one or two cases even renewal of boilers. In fact it was seldom that a shovel was sent to the shops for repairs short of complete overhauling. The numbers of shovels repaired per night by the field repair gangs run from 14 to 20.

The cost of repairs per steam shovel per service day from June 1, 1910, to July 1, 1912, was as follows:

Six months to July 1, 1910	$27.66
July 1, 1910, to July 1, 1911	22.21
July 1, 1911, to July 1, 1912	22.95

Transportation.

All spoil from Culebra was removed by railway. The railway consisted of three to four running tracks through the cut and connected with the Panama R. R. Service tracks connected the various shovel cuts with the running tracks and spurs from the Panama R. R. led to the various places where spoil was wasted or used for construction. Because of the volume of spoil and of the speed with which it had to be removed all running track and main track train operations were directed by a train dispatcher and assistants

The track system for the service of the Culebra Cut varied from time to time in mileage and location. Exclusive of the Panama R. R. track, it comprised track in the prism, track west of the canal, track east of the canal and tracks on dumps. Table XXVII shows the track mileage of each class for each of the five years to July 1, 1912.

TABLE XXVII.

Location—	1907-1908.	1908-1909.	1909-1910.	1910-1911.	1911-1912.
In canal prism	51.56	68.98	73.08	76.11	72.93
East of canal	30.63	29.45	25.51	37.83	32.13
West of canal	43.91	39.79	43.52	30.49	27.65
Dump track	43.84	56.10	58.14	64.73	54.75
Total	169.94	194.33	200.25	209.16	187.46

These totals of trackage give only a partial idea of the track work required. Each year many miles of track had to be renewed, laid new and shifted. An idea of the amount of this work is furnished by the approximate figures in Table XXVIII.

TABLE XXVIII.

Items.	1908-09.	1909-10.	1910-11.	1911-12.
Track removed	135	213	152.9	146.47
Track laid	160	219	215.08	227.13
Track renewed	...	10	20	....
Track shifted	...	1,534.6	1,534.6	1,430.84
Frogs & switches removed	573	556	590	638
Frogs & switches laid	612	927	1,169	1,096

The excavation and car equipment for handling spoil increased gradually to about a maximum in 1909-10 and then decreased. During the maximum year the motive power and rolling stock assigned to the Central Division, virtually to the Culebra cut, was as follows:

Cars—	Nos.
Decauville (industrial)	363
Steel, "Western"	419
Steel, "Oliver"	298
Steel, "Goodwin"	12
Steel, "Ingoldsby"	12
Total	1,104
Locomotives:	
Decauville	2
19x24 ins.	111
16½x23½ ins	24
20x26 ins.	19
Others	4
Total	159

Not all of the locomotives were used in hauling spoil and not all were in service at any one time. Table XXIX gives for the three years for which records are available the average number of locomotives working per day and their assignments.

TABLE XXIX.

Consignment—	1909-10.	1910-11.	1911-12.
Handling spreaders	7.8	6.35	5.85
Handling unloaders	10.97	10.23	10.03
Handling track shifters	4.35	3.64	2.92
Handling trains	125.50	126.24	117.0
Totals	148.00	146.46	135.77

The cost of locomotive repairs per service day for the corresponding three years was as follows:

1909-10	$6.94
1910-11	7.88
1911-12	8.57

The average number of cars loaded per day with excavated material and the largest numbers handled any one day are given in Table XXX.

TABLE XXX.

	1909-1910.		1910-1911.		1911-1912	
	Av.	Max.	Av.	Max.	Av.	Max.
Flat	2,156	2,648	2,268	2,569	2,403	2,674
Large stl. dump	193	179	325	446	320	224
Small stl. dump	1,179	1,946	1,147	2,166	973	1,948
Totals	3,528	4,773	3,740	5,181	3,696	4,896

The cost of car repairs per car per working day for the corresponding three years was as follows:

1909-10	$1.30
1910-11	0.87
1911-12	0.75

SPOIL DISPOSED.

Spoil from Culebra Cut was disposed of at waste dumps and in making construction fills at various points along the canal work. To July 1, 1912, 94,544,885 cu. yds. of material had been wasted at 55 separate places, the amount deposited at one place ranges from about 1,000 cu. yds. to 16,000,000 cu. yds. The main dumping grounds were as follows:

Tabernilla	16,077,847 cu. yds.
Gatun	3,709,116 cu. yds.
Miraflores	10,441,841 cu. yds.
Balboa	13,861,817 cu. yds
Panama R. R. fills	7,125,682 cu. yds.

The spoil disposed of at Gatun was employed in forming the enrockments for the Gatum dam, and that disposed of along the Panama R. R. was for the reconstructed line of that road. The three great spoil dumps were at Tabernilla, Miraflores and Balboa, and from 10,000 to 20,000 cu. yds. daily could be wasted at each of these dumps.

TABLE XXXI.

Class of work—	1908-09.	1909-10.	1910-11.	1911-12.	1912.
Loading, steam shovel	$0.1150	$0.1001	$0.0888	$0.0707	$0.0081
Loading, hand	.....	.3993	.3412	.2567	.3056
Drilling and blasting	.1413	.1149	.1190	.1048	.1157
Transportation	.1854	.1452	.1522	.1114	.1331
Dumps	.1344	.0911	.0657	.0541	.0473
Tracks	1190	.0838	.1001	.1014	.0885
Division office and supervision	.0163	.0114	.0150	.0120	.0142
General surveys	.0008	.0001	.0003	.0002	.....
Clearing site	.0001	0048	.0046	.00005	.0001
Division structures	.0002	.0012	.0013	0005	.0003
Drainage and sumps	.....	.....	.0052	.0038	.0011
Total division cost	.7128	.5517	.5416	.4880	.4707
General and administration	.1882	.1049	.0616	.0457	.0361
Plant, arbitrary	.1300	.1300	1300	.1000	.0395
Total	$1.0310	$0.7866	$0.7362	$0.6337	$0.5463

Generally, structural fills were made from trestles, as at the Gatun dam, using dump cars. At dumps the general method of procedure was as follows: A trestle was constructed and spoil was dumped from it until it was filled to track level, and as far out as possible on the sides. The track was then shifted right or left to the embankment edge and the trains were unloaded by Lidgerwood unloaders which plowed the spoil off the cars and onto the edge of the dump. Spreader cars were then used to plow the piled up spoil over the edge of the dump. When the spreader cars had reached the limit of the plow arms, the track was again shifted to the dump edge. For this work track shifting cars were employed: these cars were provided with booms and shifted the track by power. As indicating the speed of disposal at these dumps the following figures are given showing the amount of material dumped per day at the larger places of deposit:

Dump	1908-1909	1909-1910.	1910-1911	1911 1912.
Tabernilla, cu. yds	20,000	16,052	3,316	.
Miraflores	10,000	[illegible]	11,112	9,150
Balboa	11,000	11,997	[illegible]	12,929

The special equipment for unloading spoil at dumps comprised 14 spreaders, 36 unloading plows, 6 track shifters and 28 Lidgerwood unloaders. The cost of repairs to this plant per service day averaged as follows:

Item—	1909-10.	1910-11.	1911-12.
Unloaders	$20.55	$14.80	$14.45
Spreaders	17.30	14.77	14.24
Track shifters	6.67	2.25	2.77
Unloading plows	3.79	3.30	2.46

Excavation Records.

Culebra Cut excavation as recorded is not separable from total excavation in the Central Division but so great a part was it of the whole that output and cost records for the Central Division are virtually records for Culebra Cut. The total yearly excavation from the Central Division canal prism to July 1, 1913, has been as follows:

Year—	Cu. yds.
1904	60,107
1905	741,644
1906	1,506,562
1907	5,570,432
1908	13,459,378
1909	18,442,624
1910	17,806,111
1912	12,582,124
Total	105,685,902

There was estimated to remain on July 1, 1913, 9,280,237 cu. yds. to be excavated. Of the total yardage given above something over 72 per cent has been rock. The average and maximum monthly outputs for each year of heavy work, that is beginning with the year 1907-8, have been as follows:

Year—	Average cu. yds.	Max. cu. yds.
1907-08	973,771	1,216,264
1908-09	1,024,288	1,966,294
1909-10	1,483,843	1,987,714
1910-11	1,162,423	1,623,152
1911-12	1,419,774	1,712,225
1912-13	1,648,510	....

Excavation Costs.

The cost of excavation on the Central Division for each of the five years ending June 30, 1912, was as given in Table XXXI.

The yardage on which these costs were based are not those given in the annual yardages as tabulated above; they were as follows:

1909	19,067,777
1910	17,674,804
1911	18,522,692
1912	17,063,446

The costs of Table XXXI are as reported by the Division Engineer. Table XXXII shows for two years, 1911 and 1912, costs itemized somewhat differently by the cost keeping accountant.

TABLE XXXII.

Item.	1911.	1912.
Clearing	....	$0.0001
Drilling	$0.0503	.0535
Blasting	.0547	.0622
Loading	0492	.0492
Tracks	.1011	.0885
Transportation	.0816	.0734
Dumps	.0488	.0423
Pumps	.0038	0041
Maintenance of equipment	.0875	.0243
Plant, arbitrary	.1000	.0394
Division expenses	.0128	0145
Administration	.0162	0361
Total	$0.6361	$0.5479

The average cost of all excavation in the Central Division to July 1, 1913, based on 107,139,181 cu. yds. total, has been 71.05 cts. Of this total 93,305,975 cu. yds. have been removed from Culebra Cut. The estimate of 1906 was 53,765,000 cu. yds. at 80 cents; the estimate of 1908 was 78,042,295 cu. yds. at 75 cents.

All estimates of excavation in Culebra Division have, due to the development of slides in Culebra Cut, been increased each year of report. The original estimate of the minority of the International Board of Consulting Engineers was in round figures 53,700,000 cu. yds. The decision in 1908 to increase by 100 ft. the bottom width of the Culebra cut added 13,000,000 cu. yds. The revised estimate of cost of the canal made about that time was 78,000,000 cu. yds. The increases since have been:

June 30, 1910	7,330,525
June 30, 1911	4,676,278
June 30, 1912	4,614,925
June 30, 1913	9,280,237
Total	25,901,965

The bulk of the total increase has been due to the developments of rock slides in Culebra Cut.

Slides.

The rock of Culebra Cut is structurally weak and badly faulted. Its natural slope is from 1 on 5 to 1 on 10, and the original slope planned for the sides of the cut was about 3 on 2. As the excavation deepened leaving high unsupported banks, the rock at places sought its natural slope by sliding. These slides were due to familiar causes and in nature are no different from rock slides in numerous cuttings in the United States; their significance is no different and their only element of distinction is their immense size.

A tabulation of the principal slides which have occurred at Culebra is made in Table XXXIII.

TABLE XXXIII.—ESTIMATE OF SLIDES OUTSIDE OF SLOPE LINES FOR YEAR ENDING JULY 1, 1912, SHOWING TOTAL ESTIMATES OF MATERIAL TO DATE.

Location.	Date when slide first developed.	Cubic yards of material excavated to date.	Cubic yards of material remaining.	Area of slide in acres.
East Side—				
Opposite Bas Obispo	Sept., 1910	117,000	70,000	2.80
Opposite Haut Obispo	Sept., 1908	18,000	....	.60
West Side—				
Opposite Buena Vista	Nov., 1908	162,000	....	3.30
East Side—				
Opposite Las Casacadas	Feb., 1908	503,000	....	11.50
Opposite Las Casacadas	Sept., 1909	102,600	20,000	1.40
Opposite White House	Oct., 1908	286,000	....	4.00
West Side—				
White House yard	June, 1912	....	130,000	1.00
East Side—				
Opposite White House	Oct., 1909	413,000	180,000	5.80
Upper La Pita	Dec., 1909	20,000	....	1.70
Lower La Pita	May, 1910	30,000	50,000	.30
West Side—				
Cunette	Sept., 1910	67,000	10,000	.90
Empire	May, 1910	210,000	50,000	2.20
Culebra	Oct., 1907	6,765,000	1,300,000	63.00
East Side—				
Culebra	Jan., 1907	4,290,000	600,000	50.70
Cucaracha	July, 1907	2,890,000	230,000	47.10
West Side—				
Contractors' Hill	July, 1908	216,000	15,000	2.00
East Side—				
Cucaracha Village	Sept., 1911	57,000	100,000	3.40
Paraiso	Mar., 1907	385,000	10,000	5.70
Small slides	....	140,000	770,000	....
Total	....	16,671,000	3,595,000	208 00

The amount of material removed from slides compared with total excavation is given in Table XXXIV.

TABLE XXXIV.

Years.	Total excav. Cu. yds.	From slides. Cu. yds.	Pct. slides of total.
1904-1909	40,983,366	3,227,059	7.87
1910	17,865,808	2,649,563	14.83
1911	18,552,644	4,879,378	26.30
1912	17,143,067	5,915,000	34.50
1913	12,773,388	5,899,200	45.40

No plan of treatment of slides has proven thoroughly effective when once they have developed, except that of excavating and hauling away the material comprising the moving mass until the slide came to rest upon reaching the angle of repose of the particular material then in motion. This angle of repose varied greatly in different parts of the cut, depending not only upon the character of the material involved in the slides but also upon the angle of inclination of the strata and upon the angles at which the numerous dikes crossed the cut. At the Cucaracha slide the angle of repose is a trifle steeper than 1 on 5; near the center of the Culebra west slide the material still moved on a slope of 1 on 7. In one or two slides the sliding material continued to move on slopes of 1 on 10.

The slides have at times ceased movement and then begun moving again; the most notable example perhaps being the Cucaracha slide, which on June 30, 1912, was stationary, and began moving again on June 20, 1913, and forced some 2,000,000 cu. yds. of material into the canal prism. All of the other large slides mentioned in Table XXXIII continued during the year to July 1, 1913. The amounts of material removed from the several large slides to July 1, 1913, and the amounts then remaining to be removed were:

Name.	Area.	Removed.	Remaining.
Cucaracha	50	3,859,500	1,500,000
West Culebra	65	8,687,600	2,390,000
East Culebra	55	5,966,200	2,000,000

Stream Diversion and Drainage.

The Culebra cut forming the low point for many square miles of territory, and coinciding for a considerable distance with the Obispo River valley, would naturally collect vast quantities of water were steps not taken to prevent it. The course of the Obispo River was artificially changed, beginning at the point where it approached the cut. The total length of the new river channel as originally built was 5¼ miles, from a point on the east side of the Culebra cut, near the foot of Gold Hill, to a point clear of the cut, and finally discharging into the Chagres River. On account of slides encountered during construction work, the Obispo diversion gave way, and the flow of the river entered the cut for three days, causing inconvenience and damage. A new diversion channel was constructed with great speed. That the Obispo diversion was no small problem may be noted from the fact that in six years a total of 1,200,000 cu. yds. of excavation was necessary, of which nearly 40 per cent was in rock, and the total cost was over $1,000,000. The diversion was able to carry 6,000 sec. ft. of water. The Camacho diversion on the opposite side of the cut was similarly built. These two diversions took waters which flowed toward the Atlantic. The Rio Grande River formerly flowed through part of the area to be excavated on the Pacific side of the Continental Divide. It was similarly diverted, and a dike was constructed across the south end of the canal to prevent access of the river water. Keeping water out of the cut also kept out the silt which would inevitably have come down with the freshets. The elevation of the bottom of the cut was 40 ft., which was lower than the Chagres River, and a dam was built across the cut with its crest at elevation 73 to prevent the river from flowing into it.

The natural streams being thus prevented entering the work, it only remained to get rid of the water which originated along 8¾ miles of cut. This was done by means of centrifugal pumps at low points in the cut, which discharged the water over the dams. Excavation at a new level was always preceded by the cutting of a pioneer trench down the middle of the canal, in which all the water was collected and carried to the pumping stations. The summit during construction was at Culebra. Drainage to the south was carried to Pedro Miguel and after August, 1911, was carried through the center wall culvert of the locks. Drainage to the north was disposed of by pumping.

Electric Power Plants.

All lighting of the Panama Canal work, quarters and service buildings was by electricity. Electric power was also employed for blasting operations, for operating the lock construction plants, for shop cranes and for scores of miscellaneous uses. This current was generated in power plants located at Balboa, Empire, Miraflores, Gatun and Gorgona. The output and cost of current production for the years of record were as follows:

Year.	Kw. hours.	Cts. per kw. hour.
1909-10	7,027,675	3.17
1910-11	24,546,295	2.64
1911-12	25,785,910	2.24

Oil fuel was used at all stations. Current was generated by steam turbines at Gatun and Miraflores, by non-condensing engines at Empire and Gorgona and by condensing engines at Balboa. The current generated at Gatun and Miraflores was employed nearly entirely for power.

Air Compressor Plants.

Compressed air for drilling and other power service was supplied to the Panama Canal work by four principal plants located at Las Cascadas, Empire, Rio Grande, and Balboa. The equipment of these plants was changed and added to from time to time and except to state that all was of the best modern high power compressed air machinery, this equipment will not be listed.

The compressed air produced in 1907-8 amounted to about 275,000,000 cu. ft. of free air per month and cost from 5.3 cts. to 3.44 cts. per 1,000 cu. ft. During the fiscal years June 30, 1908, to July 1, 1912, the production and cost of compressed air was as follows:

Year.	Thousands of feet.	Cts. per 1,000 ft.
1908-09	4,935,110	3.29
1909-10	7,227,204	4.03
1910-11	8,261,199	3.24
1911-12	8,795,157	3.12

The compressed air main and service lines ran into miles and a schedule of it would serve no important purpose. All air was compressed to 100 lbs. per square inch.

Shop Facilities and Service.

One of the first tasks undertaken after American occupation of the Canal Zone was the provision of shop facilities by renovating the old French shop buildings and equipment and by adding new buildings and plant. By July 1, 1908, the system of main shops had been completed and shop service had been systematized. It consisted of main shops located at Gorgona, at Empire and at Paraiso. As the work progressed these main shops were supplemented by a chain of local repair shops. By July 1, 1910, the list of shops had reached about its maximum and was as follows:

Location.	No. employes.
Atlantic Division—	
Porto Bello	243
Cristobal dry dock	865
Gatun Locks	260
Gatun Dam	42
Central Division	
Empire	369
Pacific Division—	
Coculi	117
Balboa	385
Mechanical Division—	
Gorgona	1,184
Pedro Miguel	190
Las Cascades and Gamboa	96
Quartermaster's Department	
Lirio planing mill	44
Total	3,795

In addition to the above list, general repair shops with a force of 595 men were maintained at Cristobal by the Panama R. R. Also there were numerous small shops employing five or six men, distributed around the work when required and doing drill sharpening and blacksmithing. Finally there were several cars and barges equipped as machine shops and engaged in repairs to steam shovels and floating equipment.

The cost of repairs to steam shovels, locomotives, cars and other principal plant are given in the various tabulations of cost of work where the plant was used. Besides repair work the Gorgona Shops did a large amount of manufacturing, particularly foundry work. Table XXXV gives the amounts and cost of wire and brass castings for a number of years.

TABLE XXXV.

Year.	Iron. Lbs.	Iron. Cts.	Brass. Lbs.	Brass. Cts.
1906-07	3,599,798	3.91	202,424	18.55
1907-08	4,279,237	3.59	216,917	19.51
1908-09	4,586,342	2.9	333,416	16.51
1910-11	8,715,671	3.12	453,548	17.92
1911-12	7,476,014	2.87	350,711	18.11

The cost of repairs per unit of work for the years of record are given in Table XXXVI.

TABLE XXXVI.

Cubic yards.	1909-10. Cts.	1910-11. Cts.	1911-12. Cts.
Dry excavation	7.95	8.68	8.31
Wet excavation	7.15	4.79	6.76
Concrete	17.41	2.06	17.48
Sand	27.89	24.44	18.71
Stone	24.10	25.13	21.68
Dry fill	0.65	3.09	5.73
Wet fill	5.87	2.70	3.93

All repair shops were discontinued as fast as the progress of the work made them unnecessary. All useful equipment of these shops is to be transferred to the permanent shops being constructed at Sosa Hill in connection with the Pacific terminal plant. As planned the permanent shops proper will consist of 18 buildings for the machine, erecting, and toll shops; forge shop, steel storage shed; boiler and shipfitter shop, general storehouse, paint shop, car shop, planing mill, galvan ing plant, lumber and equipment shed, patte storage, foundry, coke shed, boiler hou roundhouse, gas house, paint house, and sa house. In addition to an office building the will be 9 auxiliary buildings.

The work at Culebra Cut was under t direction of W. E. Dauchy as division eng neer from November, 1904, to July, 19 He was succeeded by D. W. Bolich, who r tained the position until May, 1908. C July 1, 1908, the new organization of t canal work into three divisions was inau urated and Col. D. D. Gaillard was ma division engineer of the Central Divisio which included Culebra Cut. Mr. L. Rourke was made assistant division eng neer and held that position until July 1910, when he resigned and his duties we assumed by Col. Gaillard.

The Sea Level Channels

As stated in a previous section, the approach channels in the two oceans have a width of 500 ft. That on the Atlantic side has a depth of 41 ft. at mean sea-level, and that on the Pacific a depth of 45 ft. Their combined length is 15½ miles, so that nearly one-third of the entire canal is at sea-level. The total excavation for the Atlantic entrance was about 39,000,000 cu. yds., and for the Pacific entrance, including about 7,000,000 cu. yds. for the Balboa terminal basin and docks, was about 48,000,000 cu. yds. Work was begun in both entrances in June, 1907.

The French did a large amount of work at both ocean levels, but only a small portion of it was of use to the American canal because of changes in the location of the entrance channels. At the Atlantic end the change was complete, and while the French canal between Colon and Gatun was useful for a time in transporting material for lock construction, only about 182,000 cu. yds. of French excavation were of permanent value. At the Pacific end the location of the entrance channel from Balboa out into Panama Bay was changed from the French line, but about 3,250,000 cu. yds. of the French excavation were useful.

The French partially dredged a canal from Colon to Bohio, all of which, except the section below Gatun, has disappeared beneath the waters of Gatun Lake. In point of efficiency the French dredging equipment approached more nearly to the modern standard than their dry excavation plant. They had in service 37 dredges of Scotch, Holland, Belgian and American makes. Most of these were chain bucket or ladder dredges capable of excavating to a depth of from 31 to 41 ft. Each chain had from 28 to 31 buckets, each with a capacity of 4 2/5 cu. ft. The material was emptied through pipes 164 ft. long to the banks on either side. Three of the dredges were sea-going, with large buckets. The French had also five suction dredges, but they were not a success.

When the Americans took control they found several of the French ladder dredges in fairly good condition, and they repaired seven of them and used them during the entire period of construction. They were well built and did excellent service according to their size and capacity. The dredge "Corozal," made to order for the commission, has engines of 1,893 HP.; its chain carries 39 buckets, those for soft excavation having a capacity of 54 cu. ft., and those for hard material 35 cu. ft. It can excavate to a depth of 50 ft. Compared with the sea-going French dredge of the same type, the "Corozal" has buckets of five times the capacity, can excavate 10 ft. deeper, and can dump either into a barge or into its own bins. Its full capacity, working under favorable conditions, is ten times that of a French dredge of the same type. Its average output is easily 20,000 cu. yds. in a 24-hour day.

Most of the excavation in the approach channels in both oceans was done by two powerful sea-going suction dredges, each with an average capacity of about 22,000 cu. yds. in a 24-hour day. These dredges are built like ocean steamships, which in outward appearance they resemble, and their crews, numbering about 60 men, eat and sleep on board, working in shifts day and night throughout the week, resting only on Sundays. There were no dredges in the French equipment at all comparable to these. In all the Americans had in service during the period of construction 20 dredges of varying capacities.

An additional equipment for use in emergency work in the Culebra cut and permanent work after the canal is in operation was ordered by the Canal Commission in February, 1913, to be delivered at the close of 1913 and the beginning of 1914. This includes two dipper dredges with dippers of 10-cu.-yd. capacity for use in rock, and others of 15-cu.-yd. capacity for use in earth. For service in connection with them there will be six dump barges, each with a capacity of 1,000 cu. yds.

Long breakwaters have been constructed near the approach channels in both oceans. One in Limon Bay or Colon Harbor, called the West Breakwater, extends into the bay from Toro Point at an angle of 12° 53′ northward from a base-line drawn from Toro Point to Colon light, and is 11,526 ft. in length, with a width at the top of 15 ft. and a height above mean sea-level of 10 ft. It will contain approximately 2,840,000 cu. yds. of rock, the core being formed of rock quarried on the mainland near Toro Point, armored with hard rock from Porto Bello. Work began on the breakwater in August, 1910.

A second breakwater, known as the East Breakwater, is to be constructed. It will be without land connection, about one mile in length and will run in an easterly direction at nearly a right angle with the canal channel. It will be so placed that the opening between its end and that of the West Breakwater will be 2,000 ft.; that is, the end of each breakwater will be 1,000 ft. away from the center of the channel. The purpose of the West Breakwater is to protect the harbor against "northers," very severe gales which are likely to blow from October to January. The purpose of the East Breakwater is to prevent silting in the canal channel, which has been found to be very heavy. The cost of the West Breakwater is about $7,500,000 and of the East is estimated at about $1,000,000.

The breakwater at the Pacific entrance runs from Balboa to Naos Island, a distance of about 17,000 ft. or a little more than three miles. It lies from 900 to 2,700 ft. east of, and for the greater part of the distance nearly parallel to, the axis of the canal prism. varies from 20 to 40 ft. in height above mean sea level, and is from 50 to 3,000 ft. wide at the top. It contains about 18,000,000 cu. yds. of earth and rock, all of which was brought from Culebra Cut. It was constructed for a two-fold purpose; first, to divert cross currents that would carry silt material from shallow harbor of Panama into canal channel; second, to furnish rail connection between the islands and the mainland. A railway track and a driveway will be constructed on the top for the entire distance betwee Balboa and Naos Island. Work was begu on it in May, 1908, and on Nov. 6, 1912, th last piles were driven connecting Naos Islar with the mainland.

Dredging operations at the Atlantic entran were under the direction of F. B. Maltb from the beginning of American work unt April, 1907, when, together with those at th Pacific entrance, they passed under the dire tion of Major D. D. Gaillard, head of th Department of Excavation and Dredging. I July, 1908, when the new organization of th work into three great divisions—Atlanti Central, and Pacific—went into effect, dredg ing at the Atlantic entrance passed under th direction of Colonel Sibert, and that at th Pacific entrance under the direction of S. Williamson. In immediate charge, und Colonel Sibert, of dredging at the Atlantic e trance, was Major Chester Herding, who he it till Feb. 27, 1913.

On the Pacific side W. G. Comber, who resident engineer was in charge of dredgi under Mr. Maltby in the Atlantic entran from Aug. 14, 1905, till Feb. 1, 1907, when was transferred to the Pacific entrance, w continued in charge as resident engineer und Mr. Williamson. On May 1, 1913, Mr. Con ber, by order of the chairman and chief e gineer, was placed in charge of all dredgi operations on the Isthmus.

While the two sea-level sections of t Panama Canal were constructed by essentia the same methods, it is better to consid them separately and to follow the wo chronologically.

Atlantic Sea Level.

Excavation in the Atlantic Sea Level S tion included a considerable amount of stea shovel work through the Mindi Hills j north of Gatun, but the great bulk—fully nir tenths—of the excavation was subaqueous.

Dry Excavation.—As shown by the gene profile, Fig. 7, the Mindi Hills north of Gat rose considerably above sea level. It was cided to start the excavation through t elevation with steam shovels and as soon sea level was reached to substitute dredg Steam shovel work was begun in July, 19 two shovels working until July 1, 1908. Th shovels excavated to the date named 536, cu. yds. at a cost of 12.41 cts. per cubic ya As the cut deepened to sea level it was fou that the seepage was small and that by bui ing some small dikes, water could be k out and the excavation continued dry for much longer period than had been anticipat To July 1, 1909, some 1,152,105 cu. yds. h been excavated at a cost of 67.3 cts per cu yard. Excavation continued and to July 1912, the outputs and costs were as follov

Year.	Yardage.	Cts. per cu.
1909-10	321,716	69.88
1910-11	280,205	65.07
1911-12	421,872	63.58

Excavation by steam shovel at Mindi was frequently interrupted by flood wa which increased costs. The total of 2,181, cu. yds. excavated to July 1, 1912, cost 66.67 cts. per cubic yard.

Wet Excavation.—Dredge work on the Atlantic Sea Level Section was begun almost with the beginning of American occupation of the Canal Zone. On July 1, 1909, the dredging fleet consisted of 1 sea-going suction dredge, 2 dipper dredges with 5-cu-yd. dippers, and 3 old French ladder dredges which had been renovated. The yardage dredged to July 1, 1909, was 13,188,123 cu. yds. and its cost had been 22.2 cts. per cubic yard. This yardage included some 427,000 cu. yds. of rock which was taken out during the last 12 months. On June 30, 1909, three miles of the Atlantic Sea Level Channel had been completed.

During the 12 months to July 1, 1910, the dredging fleet remained essentially the same size as during the previous year, but a drill boat was added. The drill boat consisted of an old French hull on which were mounted eight well drills, placed four on each side of the hull, which was 22 ft. wide. The drills in each side now were spaced 15 ft. apart. This boat drilled and fired one round of holes per day. The total dredge excavation of the year was 4,556,375 cu. yds. of earth and 399,285 cu. yds. of rock at an average cost of 23.60 cts per cubic yard, not including administration charges.

The dredging fleet was not changed during the year ending June 30, 1911, and it excavated during the year 4,516,369 cu. yds. of earth and 487,038 cu. yds. of rock at an average cost of 22.18 per cubic yard, not including administration charges. On Feb. 24, 1912, the dike which had shut the water off from Mindi Cut was removed and two dredges were floated in, and after the date named the fleet consisted of seven dredges. To July 1, 1912, these dredges removed 4,870,827 cu. yds. at a cost of 24.8 cts. per cubic yard.

All the quantities named above are yardages removed from the canal prism. The dredge fleet performed a large amount of subsidiary work in connection with dock and terminal work. A summary of the dredging from the canal prism to July 30, 1912, is as follows:

Year.	Yardage.	Cts. per cu. yd.
To July 1, 1909	13,188,123	22.2
1909-10	4,955,660	26.3
1910-11	5,828,345	24.3
1911-12	4,870,827	26.9
1912-13	6,483,408	20.9

Summary.—Besides the dry excavation costs given above record is given by items of dry excavation cost for the two years ending July 1, 1912, in Table XXXVII.

TABLE XXXVII.

Item.	1910-11.	1911-12.
Cubic yards	280,305	424,872
Drilling	$0.0337	$0.0174
Blasting	.0401	.0494
Loading	.0679	.0564
Tracks	.0805	.0487
Transportation	.1209	.1280
Clearing	.0013	.....
Pumps	.0323	.0202
Maint. equipment	.0848	.0754
Plant arbitrary	.1190	.1814
Division expenses	.0202	.0183
Administration	.0497	.0406
Total	$0.6507	$0.6358

Detail costs of dredging for the two years, June 30, 1910, to July 1, 1912, are given in Table XXXVIII.

TABLE XXXVIII.

Item.	1910-11.	1911-12.
Pipe lines	$0.0074	$0.0127
Dikes	.0729	.0127
Clearing	.0010	.0006
Seagoing Suction Dredge:		
Operation	.0335	.0378
Repairs	.0117	.0259
Ladder Dredges:		
Operation	.0876	.0772
Repairs	.0620	.0790
Dipper Dredges.		
Operation	.0999	.1111
Repairs	.0317	.1437
Pipe Line Dredges		
Operation	.0718	.0424
Repairs	.0100	.0705
Tugs, Clapits and Scows		
Operation	.0777	.0811
Repairs	.0578	.0698
Drilling	[illegible]	.0897
Blasting	.0986	.1321
Small boats	.0024	.0024
Repairs misc. equipment	.0014	.0018
Plant arbitrary	.0592	.0478
Division expenses	.0093	.0077
Administration	.0218	.0207
Total	$0.2433	$0.2688
Earth excavation	5,341,307	42,955,990
Rock excavation	487,038	574,837

The total dredging to July 1, 1912, was 28,842,955 cu. yds. and the average cost was 24.12 cts. per cubic yard. Adding the yardage for the next 12 months to July 1, 1913, the total is 25,326,363 cu. yds. There remained on July 1, 1913, 1,387,000 cu. yds. of earth and 99,600 cu. yds. of rock.

Pacific Sea Level.

The excavation of the Pacific Sea Level Section was practically all subaqueous earth and rock excavation and by years of record consisted of the following quantities:

Year.	Yardage.	Cts. per cu. yd.
To July 1, 1909	16,180,107	22.9
1909-10	6,857,223	26.4
1910-11	5,549,642	27.6
1911-12	3,884,287	20.8
1912-13	4,321,956	32.4

This was all dredge work; in addition there was a considerable volume removed by hydraulic pumping, as will be described.

On June 30, 1910, when the work had reached full activity, the plant at work on the Pacific Sea Level Section consisted of one 20-in. sea-going suction dredge, one 5-cu.-yd. dipper dredge, four French ladder dredges, one drill boat and one Lobnitz rock breaker, besides the hydraulic excavating plant. The work consisted of excavating earth dredging rock and also of excavating rock where the ledge rose above the bottom of the channel. Both earth and rock dredging followed the usual methods and the main features to be noted are the methods employed to break up the underlying rock. There were three: Well drilling through the overlying earth into the rock, drill barge drilling and rock breaking, using a Lobnitz rock breaker.

Well Drill Performance.—Holes were drilled through the overlying earth, using casing and then into the rock. The operations of well drills for the two years of record to July 1, 1911, are given in Table XXXIX.

TABLE XXXIX.

Item.	1909-10.	1910-11.
Average number drills	10.5	12
Earth drilled, feet	19,756	50,889
Rock drilled, feet	33,203	35,938
Total feet drilled	52,959	86,827
Feet drilled per drill	4,472	7,236
Pounds dynamite used	237,727	345,865
Yardage of rock broken	251,812	274,339

Drill Barge Performance.—Drill barge performance records cover a period of three years to July 1, 1912, and are given in Table XLI. The drill barge had a strongly constructed steel hull, 112 ft.x36 ft. 8 ins., with two longitudinal and six transverse watertight bulkheads, dividing it into 21 compartments. Two of these amidships were used as water tanks and in others were located six fuel-oil tanks of 40 barrels capacity each. Timber spuds 24 ins. square were located at each corner of the barge, each being operated by an independent engine with rack and pinion. The barge was maneuvered by means of four manila ropes attached to kedge anchors. Three drill frames, 38 ft. high, were located along one of the gunwales and arranged to move lengthwise of the barge on rails, the movement being accomplished by an endless chain operated by hydraulic power. Each frame carried a slide to which was attached a 5½-in. rock drill. Each slide was operated by a hydraulic ram and could be moved vertically through 10 ft. Drill bars of unusual length were necessary owing to the great range of tide. The frames were supplied with steam from a central boiler through slip and swing jointed pipes. The drills could operate over a distance of 85 ft. from position of the barge. The holes were spaced 5 ft. apart on lines 6 ft. apart and were located by means of ranges ashore. The barge began operations in March, 1910, its performance from that date to July 1, 1912, was as recorded in Table XLI.

Lobnitz Rock-Breaker Performance.—The rock breaker is a device for breaking subaqueous rock by impact; the machinery was mounted on a steel hull, 100x28x8 ft., divided into water-tight compartments, some of which were used as fuel oil and water tanks.

The ram or cutter was a heavy compressed-steel spar fitted with a hardened-steel conical point and was alternately hoisted and allowed to drop by its own weight. The range of tide and depth of channel required the use of three sizes of ram, viz., 30, 40 and 56 ft. long, weighing, respectively, 15, 16 and 19.5 tons. The hoisting was accomplished by means of a 2-in. steel cable attached to the top of the cutter and passing over a large sheave, supported vertically over the ram by an A-frame 65 ft. high; from the latter the cable leads to the drum of a powerful steam winch located on the deck and especially designed for continuous running. The winch was provided with automatic gear and a lubricated steel friction clutch, which allowed the cable to follow the fall and hoists the cutter immediately after the blow was struck. The barge was maneuvered by a steam winch especially designed for paying in or out six wire ropes or chains, and was provided with quarters for the crew and an electric-light plant. Steam was supplied by a Scotch marine boiler.

After a rock shoal had been exposed by dredging the general practice was to attack the surface with the rock breaker at intervals of 4 ft. each way, the points of attack being located by ranges ashore and permanent marks on the deck. The cutter was operated in one position until the limit of penetration, which averages 3.12 ft., was reached. After the entire area of the shoal had been gone over the rock breaker was removed and the broken rock dredged out, the operation of alternately breaking and dredging was repeated until the required depth was obtained.

The performance of the rock breaker for the three years of record is given in Table XL.

TABLE XL.

Item.	1909-10.	1910-11.	1911-12.
No. holes	15,961	42,110	36,798
No. blows	106,256	269,239	172,289
Blows per hole	6.66	6.4	4.68
Penetration per blow, ft.	0.47	0,664	0.79
Penetration per hole, ft.	3.12	4.24	3.59
Area covered, sq. ft.	266,230	648,033	563,617
Yardage dredged	25,515	49,266	77,156

Summary of Dredging Operations.—Itemized records of dredging operation costs on the Pacific Sea Level Section are given in Table XLI for two years. Of the 4,321,956 cu. yds. dredged in the year to July 1, 1913, 1,047,929 cu. yds. were rock. There remained on July 1, 1913, to complete the section, for removal, 1,847,774 cu. yds. of earth and 1,600,000 cu. yds. of rock excavation.

TABLE XLI.

Item.	1910-11.	1911-12.
Clearing	$0.0006	......
Seagoing Suction Dredge:		
Operation	.0419	$0.0485
Repairs	.0150	.0247
Small Ladder Dredges:		
Operation	.0611	.0788
Repairs	.0352	.0315
5-Yard Ladder Dredge:		
Operation	.....	.....
Repairs	.....	.....
Dipper Dredges:		
Operation	.0701	.1160
Repairs	.0670	.0983
Tugs, Clapits and Scows:		
Operation	.0603	.0675
Repairs	.0314	.0232
Drilling	.1142	.0010
Blasting	.1696	.0040
Operation drill barge	.2168	.2869
Operatiion rock breaker	.0770	.0381
Small boats	.0031	.0040
Repairs, misc. equipment	.0078	.0085
Plant arbitrary	.0759	.....
Division expenses	.0082	.0056
Administration	.0245	.0176
Total	$0.2764	$0.2076
Earth excavation	5,348,530	3,474,071
Rock excavation	301,112	410,216

Hydraulic Excavation.—For a distance of about 1½ miles south from the Miraflores locks rock was found in the channel to be excavated, at an average elevation of minus 30 ft. The estimated quantity was 1,503,620 cu. yds., which was covered by 8,158,133 cu. yds. of alluvial material averaging 38 ft. in depth. It was manifestly impracticable to remove the earth and rock by dredging and subaqueous methods, as the requisite number of dredges, barges, drill scows and rock breakers would have called for an unwarranted expenditure and could not have been assembled and placed in commission so as to complete the work in the allotted time. A careful study of the conditions showed that the rock could be excavated by steam shovels, in the dry, in a shorter time and at less cost than by any other method. This, however, was not true in regard to the overlying alluvial deposit, as steam shovels and other equipment were not available in sufficient number to remove it at the required rate, and in addition the maintenance of tracks would be difficult and expensive in this partly saturated material.

A hydraulic excavating plant was, therefore, selected as being the cheapest and most expeditious method of handling the loam, especially as it could be used to redeem about 450 acres of adjacent swamp belonging to the commission. Two principal operations were involved; disintegrating the material and washing it to sumps by means of water jet under high pressure, and lifting and conveying it through flumes by means of dredging pumps. The plant consisted of the following parts:

(1) A central pumping station; (2) pipe lines and hydraulic monitors; (3) dredging pumps. The dredging pumps were mounted on reinforced concrete barges. The planned mode of procedure was described as follows:

> A portion of the area to be excavated was originally occupied by the bed of the Rio Grande. The river has been diverted and a dike will be built across the south end to prevent the tidewater from flowing up the old bed. Upon the completion of the dike the water remaining in the inclosure will be pumped out until just enough remains to float the barges in the lowest places. The giants will first be operated in the immediate vicinity of the barges so as to lower them to bed rock, thus forming a sump for the suctions of each dredging pump. The regular operation of undercutting and washing the material to the dredging pumps by means of the monitors will then begin, the cutting being extended until there is insufficient slope to sluice the material to the dredging units; the water will then be allowed to rise high enough to float the barges to new positions.
>
> The monitors will have a pressure of 130 pounds per square inch at the nozzle, and there will be three excavating units, consisting of a dredging pump supplied by either one or two monitors as may be found most advantageous for handling the material. The dredging pumps will discharge into flumes and by means of the latter the material will be distributed over the adjacent swamp lands. About 1,000,000 cubic yards will be pumped into the west dam at Miraflores to form the impervious core, and the elevation is such that a booster pump will be required, which has already been installed. If found advisable after the completion of the dam this pump may be placed on a barge to form a fourth unit for the channel work.

When work was begun numerous large boulders and sunken logs were encountered in "sinking" the barges. These obstacles, combined with the existence of rock requiring blasting at a higher level than the borings indicated, prevented the barges from settling to the grades desired, and in some instances injured the bottoms. For this reason the barges were abandoned and the dredging pumps placed at intervals along the axis of the channel with their suctions in sumps extending slightly below final grade. Two of the pumps were installed in this manner and the third was still operated from a barge.

The cost of hydraulic excavation by this plant from the beginning of work in March, 1910, to Dec. 1, 1912, when work was stopped was 69.59 cts. per cubic yard for 1,549,901 cu. yds. excavated. At this cost the entire plant cost of $432,841 was absorbed.

Terminals.

The act approved Aug. 28, 1902, authorizing the construction of the canal directed the President to "also construct such safe and commodious harbors at the termini of said canal as shall be necessary for the safe and convenient use thereof." The estimate of the cost of the canal, prepared in December, 1908, made provision for the construction of the necessary breakwaters, but did not include anything for such harbor improvements as may be classed as terminal facilities, which had been operated and provided heretofore by the Panama Railroad Co. in connection with the handling of its commercial and other business. Early in the progress of the work it was apparent that the terminal facilities required by the Panama Railroad Co. would not be adequate for the probable needs of shipping that would use the canal, and, in view of the fact that the savings on the estimates would probably enable their construction as a part of the canal work, this was advocated in 1910. Action was taken by Congress in the act approved Aug. 24, 1912, authorizing the President to "establish, maintain, and operate, through the Panama Railroad or otherwise, dry docks, repair shops, yards, docks, wharves, warehouses, storehouses, and other necessary facilities for the purpose of providing coal and other materials, labor, repairs, and supplies for vessels of the Government of the United States, and, incidentally, for supplying such at a reasonable price to passing vessels.". The sundry civil act approved Aug. 24, 1912, made the necessary appropriations for the work. While, in anticipation of favorable action by Congress, some preliminary work was undertaken, active operations could not be begun until the fall of 1912; consequently the terminal facilities cannot be completed by the time the canal is ready for passing vessels.

Main Coaling Plant.—The main coaling plant at the Atlantic terminus of the canal will be located on the north end of the island, opposite Dock No. 11 at Cristobal. The plant will be capable of handling and storing 200,000 tons of coal, with a possible increase of 50 per cent. Subaqueous storage will be provided for 100,000 tons. This plant will have railroad connection with the mainland over a bascule bridge crossing the French Canal, about half a mile south of the entrance to the old Cristobal dry dock, which connection the Panama Railroad Co. is now preparing to make.

Subsidiary Coaling Plant.—The subsidiary coaling plant at the Pacific terminus will be located at Balboa, at the outer end of the southeast approach wall of the dry dock, having a frontage of 500 ft. thereon especially adapted for discharging vessels, and a frontage of 500 ft. with elevated bunkers adapted for loading vessels and barges farther south on the site now occupied by the Panama Railroad wooden dock. This plant will be capable of handling and storing 100,000 tons of coal, with a possible increase of 50 per cent. Subaqueous storage will be provided for 50,000 tons of coal. The equipment contemplated for this plant includes the four berm cranes which have been used in the construction of the Pacific locks.

Fuel Oil Supply.—In addition to coal, facilities will be provided for at Cristobal and at Balboa for supplying shipping and the canal with fuel oil. Arrangements are being made to secure an initial storage capacity of 80,000 bbls. at each point. These fuel oil plants will be capable of increase as required.

Wharves and Piers.—The wharves and piers for commercial use at Cristobal will be located between Cristobal Point and the canal channel and behind a mole and breakwater built out from Cristobal toward the Canal Zone and Panaman waters. All wharves and piers will be of permanent construction. The five piers for which room has been provided will be about 1,000 ft. in length and 209 ft. wide, with 300-ft. slips between. To meet its own commercial requirements the Panama Railroad during the past year has started the construction of one pier with a 200-ft. slip on each side, one 1,000-ft. wharf, together with the necessary length of mole and breakwater. The head of each slip, for its full length, will be made suitable for the landing and mooring of small boats. The construction of additional piers will be deferred until the necessity therefor develops.

On the Pacific side the piers for commercial use at Balboa will be placed at right angles to the axis of the canal, with the ends of the piers about 2,650 ft. from the center of the 500-ft. canal channel. The piers will be about 1,000 ft. long and 200 ft. wide, with 300-ft. slips between the piers, and landings for small boats at the head of each slip for the full width between piers. All dimensions accord with those of the Atlantic terminal piers. There will be ample room for the construction of five piers should the necessity therefor ever develop. The construction of one pier is to be undertaken at first. In addition to the berthing space thus afforded, there will be the old French steel wharf, about 1,000 ft. in length, and also from 2,000 to 3,000 ft. of berthing space fronting the shops, which ordinarily may be considered available for commercial use when thus needed.

Superstructure, Wharves and Piers.—The superstructure of the commercial piers and wharves will consist of one-story steel sheds having a clear height of 25 ft. To assist in unloading and loading vessels, the sheds will be provided with a horizontal longitudinal girder on each side of the building, elevated to the proper height above the eaves for attaching blocks. The sheds will be of fireproof construction, either reinforced concrete or steel completely incased in concrete. The pier sheds will cover the entire piers except for a space of about 18 ft. along each side and the outer end of the pier. The wharves adjacent to the repair shops at Balboa will not be provided with any sheds. The approved arrangement of tracks on the docks includes a track at the floor level along each edge of piers and wharves. Through the center of the pier sheds there will be two depressed tracks, bringing the car floors level with the floors of sheds.

Cargo Handling Appliances.—Considerable study has been given to the convenient and economical handling and storing of freight at the terminals. The opinion is general that after the opening of the canal very little freight will be carried across the isthmus by rail. The amount of freight that will be handled from ship to ship at the terminals, or requiring storage, and the character of same are at present unknown. The best type of cargo handling appliances can not be determined until these two factors are known. It has been decided therefore that the furnishing at present of any cargo-handling cranes or similar appliances shall be deferred until the future demonstrates that they are essential and also what special type will best meet the conditions. At Cristobal, with a range of tide of scarcely a foot, freight requiring transfer can be easily handled by vessels' cargo booms, supplemented by blocks attached to the elevated girders along the sides of the pier sheds. Single pieces too heavy to be thus transferred will be taken care of by a floating crane.

At Balboa, where the average range of tides is close to 13 ft., there will be eight new electric cranes on the old French wharf and the cranes now in use on this wharf. A floating crane will be available for heavy loads. The transfer of freight from point to point on a pier may be best made by hand or electric trucks. Transfers between piers may be made in standard-gage cars or by electric trucks. The clear space which has been left along the side of each pier is sufficient to permit the installation of cranes or other appliances should the necessity for same develop in the future.

Docking Facilities.—The main docking facilities have been located at Balboa, which is in accord with the wishes of the Navy Department. There will be one main dry dock capable of accommodating any vessel that can pass through the canal locks. This dock will have a usable length of 1,000 ft., a depth over the keel blocks of 35 ft. referred to mean sea level, and an entrance width of 110 ft. The entrance will be closed by miter gates similar to the lock gates. A suitable site has been found under the protection of Sosa Hill. The dock will be founded on rock, and the excavation in the rock will be lined with concrete. Diamond borings taken over the entire area show the rock to be a solid mass of dark fine-grained andesite, which has welled up from the depths of the earth as a core of cooled volcanic rock. The rock indicated by all of the borings is strong, solid, and well suited to give a solid foundation for any structure that may be built upon it. The dry dock will be served by a 40-ton locomotive crane with a travel along both sides.

For smaller classes of vessels an auxiliary dry dock will be provided, for which a suitable foundation on similar hard rock has been found. The latter dock will be in lieu of the marine railways at first contemplated. This dock will have a usable length of 350 ft., a width of entrance of 71 ft. at mean sea level, and a depth over the keel blocks of 13½ ft. referred to the same level. The auxiliary dry dock will be provided with a floating caisson. The 40-ton locomotive crane will serve the auxiliary dry dock as well as the main dry dock.

On the Atlantic side it is proposed to retain the old French dry dock, which has a width of entrance of 50 ft., a usuable length of 300 ft., and a depth over the sill of 13 ft. referred to mean sea level. It was the opinion of the board appointed to consider the matter that the commercial requirements now in sight would not warrant the expenditures necessary for the construction of a dry dock at Cristobal capable of taking the largest vessel, in view of the dry dock at Balboa, less than 50 miles away, to which any large vessel on the Atlantic side of the canal requiring to be docked and which would not otherwise have to pass through the canal, could be taken and returned without the payment of canal tolls.

Repair Facilities.—The main repair shops will be built at Balboa within the area between the dry dock and the repair berths and to the northeast of same. They are designed to maintain the following equipment:

(a) Lock, spillway and power-plant machinery.

(b) Water and land equipment retained for the maintenance and operation of the canal.

(c) Rolling stock and equipment of the Panama Railroad.

(d) Mechanical apparatus connected with the coaling plants, fortifications, cold-storage plant, wireless stations, etc.

(e) The making of such repairs, etc., as may be required by private parties and corporations on the isthmus.

(f) The making of repairs, etc., required by commercial vessels.

(g) The making of such repairs as may be required by vessels of the United States Navy and vessels belonging to other Governments.

The main metal-working shops, including machine shop, erecting and tool shops, the forge shop, and the boiler and ship fitter's shop, together with the shed for the storage of steel, have been placed end on between the dry dock and repair berths. The general storehouse, foundry, woodworking shops, subsidiary buildings, and office building have been placed parallel to the line of the dry dock and water front and northeast of the main shops. Two lines of railroad tracks run past each end of the main metal-working shops, and one track through the center of same. The main shops will be provided with overhead traveling cranes, the crane railways being extended through each end of the buildings over the railroad track. Material brought out of the southeast end of the main shops by the overhead cranes can be handled directly into the dry dock by the 10-ton crane, or loaded into cars. The arrangement aimed at has been to secure the most economical handling of material and work to and from the shops. [illegible] possible machines now in use will [illegible] in the new shops, which will be [illegible] with individual and group drive being used. The floor area of the principal buildings is as follows:

Building.	Floor area, square feet.
(a) Machine, erecting and tool shops	67,420
(b) Forge, pipe fitters', tinsmiths' and copper shop	31,650
(c) Steel storage shed	18,080
(d) Boiler and shopfitters' shop	45,940
(e) General storehouse	89,920
(f) Paint shop	12,760
(g) Car shop	38,800
(h) Joiner, carpenter and pattern shop	48,240
(i) Galvanizing shop	5,620
(j) Lumber and equipment store shed	67,060
(k) Steel, iron and brass foundry	37,060
(l) Coke shed	3,070
(m) Boiler house	2,380
(n) Pattern storage building	13,870
(o) Office building	9,500
Total	491,370

There will be a number of subsidiary buildings the dimensions of which have not yet been determined. These buildings are being designed as modern shops of permanent construction, with steel frames. The roofing will be of permanent character. The sides and ends will be left open for ventilation and light, protection from sun and rain being afforded by overhanging sheds.

Until future requirements are more definitely developed, it is proposed to retain existing shop facilities of moderate extent for emergency repairs in the vicinity of the old French drydock at Christobal.

The flooding of Gorgona shops by the rising waters of Gatun Lake about July 1, 1913, will render it necessary to make the new buildings ready for occupancy at that time.

Floating Equipment.—For the expeditious and convenient handling of lock gate leaves, as well as for commercial and other canal needs and general wrecking purposes, it will be necessary to provide a floating crane of the largest practicable capacity at each terminus of the canal. An investigation is now under way to determine the best way in which these requirements can be filled, with a view to purchasing two floating cranes.

For handling vessels of the largest size at Cristobal and at Balboa, harbor tugs of high power will be required, and the tugs now owned by the commission will not be satisfactory or economical for this service. It is proposed to provide two large harbor tugs at each terminus of the canal for this purpose.

As a great deal of coal may be expected to be furnished to shipping in barges or lighters that can be laid alongside vessels, it will be necessary to equip the canal with a sufficient number of barges, as well as fuel, oil, and fresh-water lighters. It is expected that a large number of steel barges of large capacity, which have been used by the commission on construction work, will be available for transfer to this service. A barge capacity of 16,000 tons of coal has been recommended by the Navy Department for the Atlantic terminus and 8,000 tons at the Pacific terminus.

If the canal business warrants it, a tender for passengers and mail will be provided at each terminus.

During the past year work has been proceeding on the general and detailed plans of terminals, with a view to starting work as soon as it is authorized and appropriations become available. In anticipation of favorable legislation, at the latter part of the year some preliminary work was undertaken, in order not to lose time and to keep the plant and organization employed, including grading and dredging at Balboa.

Permanent Quarters.—During the past year the locations of the permanent administration building and canal headquarters and the permanent settlements for employes have been determined after investigation and consideration by committees representing the different interests concerned. The permanent administration building, as approved May 21, 1912, will be on a knoll west of Ancon quarry and immediately east of the railroad track leading to Ancon quarry, where it will overlook both the terminal piers and the canal entrance. The quarters for employes attached to the administration building will be erected in the general area adjacent to and northeast of the administration building. Employes connected with the shops, docks, and other terminal facilities will be housed in quarters erected in the area surrounding the slope of Sosa Hill and on the fill adjoining the Ancon-Balboa highway.

There will be a permanent settlement at Pedro Miguel for employes of the Pacific locks, and one at Gatun for employes of the Atlantic locks. The settlement at Cristobal will be maintained and possibly the settlement at Ancon. No necessity is apparent for any other than the above five settlements, except for the military forces which may be stationed on the Isthmus.

There will be a number of subsidiary buildings the dimensions of which have not yet been determined. These buildings are being designed as modern shops of permanent construction, with steel frames. The roofing will be of permanent character. The sides and ends will be left open for ventilation and light, protection from sun and rain being afforded by overhanging sheds.

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Aids to Navigation.

To guide navigators in taking ships through the canal there is provided a complete system of lights, beacons and buoys so arranged as to delineate the channel clearly both at day and at night. The Atlantic entrance is marked by a light of the fourth magnitude placed on the end of the Toro Point Breakwater, where there is also a compressed-air fog whistle and a submarine bell. The tangents in the canal are defined by range lights. Vessels going in opposite directions use different ranges giving courses 250 ft. apart. There are also side lights, spaced about one mile apart on each side of the channel. In the Culebra cut the range lights are omitted, due to the tangents being so short and the banks too steep for placing them. There are, instead, 35 concrete beacons at tangent points and at intermediate locations. By a system of screening, only those lights will be visible to the navigator which are necessary to define the channel where the ship may be. The sides of the canal will be further marked by acetylene, flashing gas buoys of 450 candlepower, and intermediate spar buoys.

The range lights involved much clearing of forest growth, since the troches are in general of the full width of the canal at the start, narrowing down to 480 ft. at the rear light.

All lights, beacons and buoys are white, and are distinguished from one another by characteristic flashes. Gas and electricity are used for illuminant, preferably electricity where feasible. Inaccessible lights are operated by means of compressed acetylene dissolved in acetone.

The locks are illuminated by means of 400-watt tungsten lamps, on concrete posts, with concrete reflectors. The lamp is 30 ft. above the coping and is screened by concrete skirts, thus keeping the glare out of the navigator's eyes.

Sanitation and Cost Accounting

Sanitation of the Canal Zone.

When Americans took possession of the Canal Zone in 1904 the Isthmus was subject to annual epidemics of yellow fever and malarial fever was endemic the year around. About 20,000 persons had died on the Panama Canal work up to that time. The number of deaths during the construction of the Panama R. R. had been from 800 to 1,000. Report naturally exaggerated the unhealthful conditions and Panama, when the second Isthmian Canal Commission began its work, was popularly regarded as little better than a pest-hole. Almost the first work of the commission was therefore necessarily the sanitation of the Canal Zone to make it a reasonably safe place in which to live and work. The steps taken have already been outlined: a more complete general account of the work done is given here.

It had already been demonstrated that both yellow fever and malaria were mosquito borne diseases and that infection was communicated in no other way than by mosquito bites. The problem at Panama was, therefore, the problem of eliminating mosquitoes and Panama was infested by mosquitoes. Its temperature permitted mosquito propagation the year around; undergrowth and lush grass flourished; during the rainy season innumerable ponds were formed and remained for months as breeding places for anophelcs or malaria mosquitoes; domestic water supply for the dry season was obtained by storage in tanks and cisterns and furnished breeding places for the house loving stegomyia or yellow fever mosquito; seven out of ten of the natives had malarial infection; there was no adequate quarantine against fever-infected immigrants.

The main lines of Canal Zone sanitation were suggested by the conditions described and were as follows: (1) Rigid quarantine to prevent the admission into the Zone of infected persons; (2) isolation of all fever cases in doubly screened apartments to prevent infection of the mosquitoes by the sick and protection by screening all living quarters to prevent infection of the well by mosquitoes; (3) provision of adequate pure water supply and sanitary sewage and waste disposal; (4) elimination of mosquitoes from the Zone.

Quarantine. Methods of quarantine as applied at the Canal Zone call for no further mention than the statement that they corresponded to the most approved practice in the United States and were strictly followed. They have been successful.

Isolation and Housing.—Protection of both well and sick from access by mosquitoes is merely a matter of screening their living quarters. At Panama hospitals for the sick and living quarters for the well were screened and doubly screened. The only unusual features of this screening were: that it was unusually complete and that it was so constructed as to prevent removal and to require the minimum care and attention by the householder. It was so designed wherever possible that the householder had no occasion for either comfort or convenience to remove, open or interfere with the screening in any way.

Isthmian Water Supply.—The provision of a pure water supply for the Canal Zone was one of the earliest tasks undertaken by the Second Isthmian Canal Commission. A Division of Municipal Engineering was created to perform this task and to provide for sewage and waste disposal and street and road improvement. This Division having completed its main task was abolished on Aug. 1, 1908. Succeeding work of municipal engineering has been in charge of the Division Engineers.

The plan of water supply adopted was a chain of reservoirs across the Isthmus with mains leading to and distribution systems supplying labor centers, where reservoirs were impracticable or where only temporary supply was required old condensers and boilers from the French canal plant were renovated and distilled water was delivered bottled to all quarters. The list of reservoirs constructed is as follows:

Location.	Year built.	Million gallons.
Rio Grande Reservoir..........	1906	196
Ancon Hill Reservoir...........	1906	1
Cocoli Lake Reservoir..........	1909	429
Camache Reservoir	1907	375
Caribali Reservoir	1906	160
Agua Clara Reservoir..........	1910	613
Brazos Brook Reservoir.......	1906	650
Total		2,724

The Cocoli Lake Reservoir was designed for temporary supply only, is in the site of Miraflores Lake and will be submerged by the lake when the canal is completed. Its bed was left in its natural condition of vegetation. The sites of all the other reservoirs were cleared of all vegetation and debris and their watersheds were depopulated. Periodical inspection and analysis preserve the reservoir and watershed in sanitary condition.

The Rio Grande Reservoir supplies the city of Panama and the American settlements as far inland from Panama as Culebra. At Ancon a filter plant consisting of two 1,500,000-gal. pressure filters was installed to filter the supply to Ancon and Panama. These filters worked successfully. The Rio Grande Reservoir was completed in 1906 and the filters at Ancon were installed in 1906-1907. At Cocoli a temporary filter plant was built consisting of two 50,000-gal. wooden sedimentation tanks, a 7,000-gal wooden closing tank and 2,100-gal tanks for storing alum solution. The filtered water discharges into the Rio Grande main. The smaller reservoir at Camacho supplies Empire and Las Cascades; that at Caribali supplies Gorgona, and in addition there is a pumping plant supplying the small settlements from San Pablo to Frijoles.

The Brazos Brook reservoir was built in 1906 to supply Mt. Hope, Cristobal and the city of Colon, and a filter plant was installed at Mt Hope. This plant was of the same size as that at Ancon and originally was like it in make and arrangement of filters. The heavy iron organic character of the Brazos water clogged the filters at Mt. Hope and rendered them inoperative and the plant was modified by the installation of a 100,000 gal. and later a 500,000-gal. preliminary coagulation and sedimentation tank. The Agua Clara Reservoir supplying Gatun was built in 1910 and the most complete filter plant on the Isthmus is installed there. It consists of a 70x70x10-ft. settling basin, a mixing tank, four 17x17-ft. rapid gravity filters and a 200,000-gal. clear water well. The mixing tank is provided with grids for introducing the aluminum sulphate and with perforated pipes for the forced injection of air.

The water works outlined were, as will be noted, completed in 1906-7 except the temporary works at Cocoli and the new works at Agua Clara for Gatun. Naturally the distribution system and mains have been modified, added to and abandoned from time to time to meet the progress of the canal work. Also various auxiliary supplies and pumping plants have been installed along the work to meet temporary deficiencies and demands. The final system of water supply planned is in general that already described except that new filters have been installed at Mt. Hope and the Brazos Brook Reservoir has been connected by supply main with Gatun Lake. Gatun Lake will doubtless also be called upon to increase the Rio Grande supply

The natural water as impounded is soft, being practically rain water, but owing to the organic matter contained it is not entirely palatable and its appearance is not good unless filtered. The filtered water is remarkably good and the water works of the Canal Zone have been an agency for the reduction of disease second only to the work of mosquito extermination.

Mosquito Extermination.—The general procedure of mosquito extermination is well known; at Panama it was simply conducted with unusual thoroughness and persistence With the introduction of a water supply and of sewerage the sanitary department destroyed all the old water tanks and rain barrels which had afforded breeding places for stegomyia; it filled cesspools, cleaned up waste heaps, regraded open spaces and paved streets. A house-to-house fumigation was conducted. Coupled with this work of destroying the breeding places and killing of stegomyia by fumigation the medical inspectors maintained a house-to-house canvass and any person suspected of yellow fever was taken to hospital and quarantined, force being employed if necessary. With the quarantine of immigrants, these methods cleared the Canal Zone not only of yellow fever but also of the mosquito which carries this disease.

To exterminate the malaria mosquito anopheles with any such completeness was impossible. The effort of the sanitary department was, therefore, by all known means to reduce the number of malaria mosquitoes The procedure was as follows: The areas occupied by the canal construction forces, some 1,200 acres, were cleared of undergrowth and grass was kept cut to less than a foot in height for a distance of 200 yards around settlements. Swamps and ponds were drained or where drainage was impossible they were oiled or impregnated with larvicide; subsoil drainage was installed to keep the ground surface dry; in fact no possible preventive that knowledge and diligence could provide was neglected. So successful were these measures that in the dry season at Panama a mosquit was rarely seen. Malaria cases decreased over 60 per cent.

Cost and Results of Sanitation.—As the preceding outline will have indicated there has been grouped under the head of sanitation a number of activities for health preservation—sanitation proper, water supply, sewerage, garbage and refuse disposal, hospitals, quarantine, etc. About $20,000,000 will have been expended for these services by the time the canal is opened for traffic. Of this sum some $9,000,000 will be charged to hospital service and the care of the sick; $7,500,000 to general health service; $2,500,000 for water supply and sewerage, and $1,000,000 for administration.

The results of this work have been to make the Canal Zone a reasonably healthful place of abode. This conditon will continue only so long as the same rigid methods of health protection are enforced as have been enforced during canal construction.

Cost Keeping System.

The value of construction cost keeping was recognized from the first by those having direction of the work at Panama. Compilation of costs was begun July 1, 1907, and modifications in the methods were made from time to time until Jan. 1, 1910, when a system of standard cost accounts was finally adopted. An outline of this system follows:

The cost data were compiled in the division offices under the supervision of an accountant in the office of the chief engineer. By this method the information was secured at an early date and greater accuracy was insured, the work being directly under the supervision of the division engineers interested. It was not thought necessary to compile a minute detail of costs, as in a work of this magnitude the advantage of such information would not warrant the additional expense. The costs for the principal items of the work were available between the 18th and 22d of the month, and the completed cost sheets reached the chief engineer on the 25th of the month following.

As soon as possible after the close of each month, but not later than the 25th of the month following, the different departments and divisions submitted to the accounting department, furnishing copies to the chief engineer, a statement showing their expenditures for labor and material and other expenses, classified under certain general headings. The labor shown thereon must agree with the total amount of the payrolls approved for payment; the material charges with the reports of material issued by the chief quartermaster, and miscellaneous charges with the amount debited the divisions and departments by the accounting department. Carrying the same sub-division as to labor, material and other expenses, the construction divisions furnished the chief engineer's office with detail cost sheets showing the cost in total and per unit for each sub-division of their work. Abnormal fluctuations from one month to another were investigated and reports thereof made to the chief engineer

There was also furnished the chief engineer reports of the performance of the various units of construction machinery and of the stone and sand-producing plants. These reports showed the total hours in service, the hours actually engaged in useful work, the time lost by delays and the cause thereof, the total cost of work accomplished, and the average quantity produced per hour. From these reports the efficiency of the equipment and of the manufacturing plants could be ascertained as well as the economy of their operation. By comparing the performance of similar equipment in the various divisions, together with the costs reflected by the cost sheets, the relative efficiency and economy were obtained.

A large proportion of the expenses of the shops, such as supervision, power, cleaning, etc., could not be located to any particular work order and constituted what is termed "shop expenses." The percentage of these expenses to located labor in the various machine shops on the Isthmus ranged from about 30 to 63 per cent, according to the quantity and nature of the work performed. The causes for this high overhead expense were the items of holidays, sick and vacation leave. The details of "shop expenses" were closely scrutinized, as they were a measure of economy in the operation of the shops.

The unit costs were published quarterly in the "Canal Record," the official journal of the Commission, and this was found to stimulate a spirit of rivalry between the different divisions, extending to all employes, which tended to lower construction costs.

The following methods were used in compiling the cost of the units of construction on the Isthmus:

Distribution of Labor.—The classification of the labor was made by the foremen directly in charge of the work, who were provided "labor distribution books" for this purpose. Two sets of books were used, one for odd days and one for even days. These books had 16 pages, which carried the dates of the month vertically in the margin. Each page was subdivided into columns and the foreman inserted at the head of each column the various rates of pay of the men working under his supervision. A page was used for each job on which the gang was engaged during the month, and opposite the date the total number of hours worked at each rate of pay was inserted. The books used during the day were returned to the division office each night and the books for the next day issued. The distributions were scrutinized by the division accountant and by the supervisor in charge of the work and any unusual item was investigated.

At the close of the month these books were summarized, the cost of each job ascertained, and the total of the labor distribution book was balanced against the payroll of the gang. Experience showed that the difference between this distribution and the actual payroll was very small, something under 2 per cent, which difference was adjusted on a percentage basis. Labor expenses for steam shovels, locomotives, cranes, and other units of equipment were distributed to the units of the work on a basis of the time in service. This information was obtained from daily reports furnished by the man in charge of equipment, giving the number of hours at work at each different job. Labor distribution books were not used in the Central Division, the classification being easily obtained from the payrolls, as all the expenditures were chargeable to one class of work, dry excavation.

At the manufacturing and repair shops on the Isthmus the work was done on numbered orders issued by the shop office, and all expenses in connection with the order were reported under this number. When reporting for duty the workmen were given cards showing date, name, trade and rate of pay. On these cards the workmen reported their time by order numbers. When engaged on several jobs during the same day the time worked on each job was shown on the card. These cards were returned to the shop office at night and were computed and summarized so that the labor cost on any job was available at any time. At the close of the month the distribution of the cards was balanced against the actual payroll.

Material.—At the inception of the work difficulty was experienced in arriving at the true prices of material. In October, 1908, a price book was completed, covering every article in the storehouses on the Isthmus. These prices were based on the cost of the material delivered in Colon. As the bulk of the material was purchased on yearly contracts but little difficulty was had in determining correct prices. Any differences between the actual value of the stock on hand and the value shown by the records of the Commission was determined by annual inventories and adjustments made in subsequent months.

All of the material on the Isthmus was in charge of the Quartermaster's Department, and was drawn only for immediate use, with the exception of a few commodities, such as cement, lumber, track material, etc., which for reasons of economy were shipped from the dock direct to the divisions, and stored at central points to be used as needed. Material was drawn from the Quartermaster's Department on requisitions, called "foremen's orders." A number of the foremen were authorized to draw material. These orders showed the purpose for which the material was drawn and by the use of carbons three copies were secured. Two copies were delivered to the storekeeper on receipt of the material, and one copy was retained by the foreman for reference. One of the copies furnished the storekeeper was priced and extended by him, and all orders were sent to the respective division offices daily to be charged to the proper account. At the close of the month the storekeepers furnished the chief engineer's office with statements showing the value of the material issued to the various divisions and departments. The total of these statements had to agree with the total material reported used by the divisions and departments on their monthly reports, and any differences were investigated and adjusted in the accounts for the succeeding month, and statements were furnished the chief engineer's office by the divisions and departments. When material was drawn for manufacturing or repair work in the shops the foremen's orders showed the number of the shop order for which the material was required and when the priced copies were returned to the shop office they were separated under each number and summarized when it was desired to ascertain the material cost of the job, or when it was completed. Upon the completion of a manufactured article it was invoiced to the quartermaster's storekeeper at the actual cost of labor and material, plus such a surcharge on the labor cost as would cover the unlocated expenses of the shop. Expenses for repairs to equipment, etc., were invoiced monthly directly to the division interested.

In addition to the expenses for labor and material there were expenses for work done by other divisions, such as repairs to equipment, electricity, wagon service, etc. The division or department doing work for another rendered a detailed invoice and reported the amount on its monthly statement of expenditures. It was the duty of the accounting department to see that all charges of this nature were taken up by the division or department against which the bill was rendered.

The items of general expenses were prorated to the various construction divisions by the accounting department, on the basis of labor charges or material used, as the case might be. These expenses were taken up on the cost sheets by the divisions but were shown as a separate item under each unit of construction.

Manufacturers of Construction Plant and Materials

Contractors for Lock and Spillway Operating Machinery.

Some $14,000,000 of construction for lock and spillway work was done by contract. These contracts were let at different times and were for various amounts, and were allotted to a number of persons. Many of the contracts were for small amounts and were not reported. Much structural material was bought in the open market and is also not reported, and much material was produced in the Canal Commission founderies and shops. The complete list of principal contractors is given in Table XLII.

TABLE XLII.

Contractor—	Material.	Price.
United Engine & Foundry Co	Anchorages for lock gates	$ 528,124.70
Rosedale Foundry & Machine Co	Cylindrical valves	84,177.50
Chapman Valve Co	By-pass valves	4,399.89
Crane Co.	By-pass valves	6,456.00
Lynch Bros. Foundry Co	By-pass valves	9,114.90
Penn Bridge Co	Ladder rungs	2,058.40
United Engine & Foundry Co	Snubbing hooks, girders, etc	16,846.56
Penn Bridge Co	Fixed irons for rising stem gate valves and 2 valves	29,252.56
Wheeling Mold & Foundry Co	Fixed irons for rising stem gate valves	199,106.24
McClintic-Marshall Co.	Mitering lock gates	5,374,474.00
Wheeling Mold & Foundry Co	Fixed irons for rising stem gate valves	225,895.61
Rosedale Foundry & Machine Co	Cylindrical valves	140,588.00
Pennsylvania Steel Co	Snubbing hooks and girders	17,216.94
Wheeling Mold & Foundry Co	2 rising stem gate valve machines	15,299.00
Wheeling Mold & Foundry Co	114 rising stem gate valve machines	651,877.88
Wheeling Mold & Foundry Co	Spare parts	11,072.50
Wheeling Mold & Foundry Co	2 cylindrical valve machines	2,264.00
Wheeling Mold & Foundry Co	118 cylindrical valve machines	102,258.09
Wheeling Mold & Foundry Co	Spare parts	2,064.70
Wheeling Mold & Foundry Co	12 auxiliary culvert valve machines	11,103.84
General Electric Co	1 motors for rising stem valve machines	2,780.00
Westinghouse Electric & Mfg. Co	2 motors for rising stem valve machines	1,454.00
Allis-Chalmers Co.	2 motors for rising stem valve machines	3,200.00
General Electric Co	114 motors for rising stem valve machines	75,644.70
General Electric Co	4 motors for cylinder valve machines	1,355.00
Westinghouse Electric & Mfg. Co	2 motors for cylinder valve machines	3,822.00
Allis-Chalmers Co.	2 motors for cylinder valve machines	2,610.00
General Electric Co	118 motors for cylinder valve machines	11,636.30
General Electric Co	12 motors for auxiliary culvert valve machines	1,234.20
General Electric Co	2 lt. sw. for rising stem valve machines	196.00
Westinghouse Electric & Mfg. Co	2 lt. sw. for rising stem valve machines	225.00
Cutler-Hammer Manufacturing Co	2 lt. sw. for rising stem valve machines	98.00
Cutler-Hammer Manufacturing Co	114 lt. sw. for rising stem valve machines	5,586.00
General Electric Co	2 lt. sw. for cylinder valve machines	196.00
Westinghouse Electric & Mfg. Co	2 lt. sw. for cylinder valve machines	225.00
Cutler-Hammer Manufacturing Co	2 lt. sw. for cylinder valve machines	98.00
Cutler-Hammer Manufacturing Co	118 lt. sw. for cylinder valve machines	[illegible]
United States Steel Products Co	6 emergency dams, complete	2,243,068.38
United States Steel Products Co	Steel cross-ties, tie clips, conductor slot covers, conductor rails, channels, etc	135,318.71
R. C. Hoffman Co	Rack and miscellaneous castings	250,367.17
Ross-Mehan Foundry Co	Malleable iron cover plates, etc	17,678.16
Westinghouse Electric & Mfg. Co	Porcelain insulators	5,858.76
Oliver Iron & Steel Co	Steel bolts, lead for gaskets, coil springs, etc	12,054.08
Chicago Railway & Traction Supply Co	Bumping posts	2,632.50
United States Steel Products Co	Lock gate recess covers	130,000.00
Wheeling Mold & Foundry Co	2 miter gate moving machines	16,270.00
Wheeling Mold & Foundry Co	90 miter gate moving machines	653,422.50
Wheeling Mold & Foundry Co	Spare parts	7,670.56
Richard Manufacturing Co	1 miter forcing machine	1,150.00
Richard Manufacturing Co	45 miter forcing machines	28,205.00
Richard Manufacturing Co	Spare parts	1,920.50
General Electric Co	2 motors for gate moving machine	1,134.40
Allis-Chalmers Co.	2 motors for gate moving machine	3,000.00
General Electric Co	90 motors for gate moving machine	15,288.00
General Electric Co	1 motor for miter forcing machine	339.00
General Electric Co	45 motors for miter forcing machine	13,230.00
General Electric Co	Track material	7,236.57
General Electric Co	Steel rails	67,402.50
McClintic-Marshall Co.	Spillway gates, caissons, footbridges, etc.	102,686.91
McClintic-Marshall Co.	Rising stem gate valves, guard gate valves, lateral culvert valves, bulkhead gates and screens	38,994.91
Excelsior Tool & Machine Co	Fixed irons for spillways	25,771.12
Westinghouse Machine Co	Roller trains, sealing vices, rocker bearings, rollers, etc.	17,117.58
Vulcan Rail & Construction Co	Railings for spillways	[illegible]
St. Louis Screw Co	Brass bolts, nuts, etc	4,772.05
U. S. Steel Products Co	Deckings and intake bridges	21,111.97
I. B. Kendal Co	Bars for deckings	[illegible]
Shawmut Clay Mfg. Co	Vitrified tile, mandrels, etc	85,350.00
Wheeling Mold & Foundry Co	Copper conductor rail and bars	168,123.01
Clinton Wire Cloth Co	Reinforcing grids	360.00
Sneath Glass Co	Vault lights	[illegible]
Pelton Water Wheel Co	Penstocks	[illegible]
Pelton Water Wheel Co	Turbines, governors, headgates, etc	[illegible]
General Electric Co	Generators, exciters, etc	[illegible]
Niles-Bement-Pond Co	Traveling crane	7,445.00
United Engine & Foundry Co	Fender chains	128,905.00
General Electric Co	1 trial towing locomotive	[illegible]
General Electric Co	[illegible] towing locomotives	[illegible]
Standard Foundry & Machine Co	Snubbing posts, etc	[illegible]
American Car & Foundry Co	Bolts, links, eyes, etc	[illegible]
Railway Spring Co	Helical springs	[illegible]
Steacy-Schmidt Mfg. Co	Spillway gate machine and [illegible]	[illegible]
H. R. Worthington	Pumps	[illegible]
Industrial Works	Cranes, [illegible] and [illegible]	[illegible]
American Steel & Wire Co	Triangular mesh reinforcement	[illegible]
U. S. Steel Products Co	Cross-bond[illegible], punches, riveting hammers, copper cable	[illegible]
Natl. Electric Supply Co	Galvanized conduit, pipe couplings, bushings, locks, nuts, etc	[illegible]

Suppliers of Construction Plant and Materia

The total expenditures for constructio plant to July 1, 1911, when new purchases ha practically ceased, was some $27,000,000, itemized as in Table XLIII.

It is impracticable to schedule all the kind and makes of plant included in so vast a quantity, and as a matter of fact no complet published record is available for compilin such schedule, were it possible to give it space The kinds of plant that were most used i constructing the canal have been described i the preceding sections, the principal supplier of this plant are named in the succeeding paragraphs:

Dry Excavation.—The largest single ite of plant cost is that for dry excavation plan This comprised air compressors, drills, explosives and blasting supplies, and steam shovel There were 28 air compressors furnished fo the canal work by the Ingersoll-Rand Co New York; the Sullivan Machinery Co., Chicago; the Laidlaw-Dunn-Gordon Co., Cincinnati; the Janesville Iron Works, Janesville Wis., and the Westinghouse Electric & Manufacturing Co., Pittsburgh. These compressor are not listed by makers' names nor were th rock drills, some 725 of which were supplie to the canal work by the Keystone Quarr Drill Co., Beaver Falls, Pa.; the Star Drillin Co., Akron, O.; the Ingersoll-Rand Co., Nev York; the Sullivan Machinery Co., Chicago, an the Wood Drill Works, Paterson, N. J. Th Sullivan Machinery Co. also furnished som two dozen channelers for lock pit excavation The dynamite, some 25,000 tons, was furnishe by the Du Pont Powder Co., Wilmington, Del, and this company also furnished the grea bulk of blasting supplies, amounting to som $600,000 worth.

The steam shovel plant consisted of 1 shovels of all sizes. One of these was a sma revolving shovel made by the Thew Automatic Shovel Co., Lorain, O., and there wer perhaps a dozen shovels of $1\frac{3}{4}$ cu. yd. dippe size or smaller sizes. The heavy rock excavation called for shovels of large capacity an strong construction and there were nearly 8 shovels having dippers of 3 to 5 cu. yd capacity. These heavier shovels, as well most of the lighter ones, were Marion an Bucyrus shovels, furnished by the Mario Steam Shovel Co., Marion, O., and the Bucyrus Co., South Milwaukee, Wis.

Dry excavation furnished the great bulk c the work of the transportation plant, whic included 300 locomotives and 1,500 cars. Or Decauville locomotive was employed for ligh service and 101 French locomotives were renovated. The remaining motive power plar consisted of standard and narrow gage stear locomotives and electric lomocotives furnishe by the Baldwin Locomotive Co., Philadelphia the American Locomotive Co., Schenectad N. Y.; H. K. Porter Co., Pittsburgh; Westinghouse Electric and Manufacturing Co Pittsburgh, and the General Electric Co Schenectady, N. Y. A great variety of car including pay cars, industrial cars, motor an narrow gage cars, were employed besides th spoil handling cars which comprised son four fifths of the construction rolling stoc The spoil cars were furnished by the Lidgerwood Manufacturing Co., New York; Roge Ballast Co., Chicago; Goodwin Car Co., Ne York, Wm. J. Oliver Mfg. Co., Knoxvil Tenn., and Austin Manufacturing Co., Chicago.

The transportation plant included beside cars and locomotives a large number c spreaders, unloading plows and Lidgerwoo unloaders, furnished by the Lidgerwood Manufacturing Co., New York, and the O. F. Jorda Co., Chicago.

Track materials—rails, frogs, switche cross ties, etc.—were purchased in the ope market and the purchases were distribute widely among American railway supply firn and are not reported. This is also true c hundreds of items of small tools and mine

Pennsylvania Flexible Metal Tubing Co.	Flexible conduit and couplings	4,466.00
Chas. A. Schieren Co.	Leather gaskets	42.00
O. A. Danzenbaker	Asbestos pads	75.60
Detroit Screw Works	Brass pins	117.00
R. C. Hoffman Manufacturing Co.	Steel splice bars	568.82
Standard Foundry Co.	Crankgear recess cover seats	30,421.64
General Electric Co.	Starting panels, limit switches, commutating switches, etc., indicators for control machinery, switchboards and control switchboards, starting panels for chain-fender	172,658.70
Union Switch & Signal Co.	Indicating devices	450.00
General Electric Co.	Motors	110,260.18
General Electric Co. (Cameron)	Pumps	26,921.50
General Electric Co.	Float switches and starters	13,882.20
Alberger Pump Co.	Pumps	21,860.78
Earle Gear & Machinery Co.	Handrail gearings	5,360.00
U. S. Steel Products Co.	Crossovers and turnouts	44,142.00
General Electric Co.	Insulated cable	132,028.60
	Transformer room equipment (bids opened May 22, but no award made to June 30)	....
	Auxiliary electrical equipment for Gatun hydroelectric station (bids to be opened July 8)	....
McClintic-Marshall Co.	Erection of lockgate recess covers, purchased under Circular 620	30,000.00
	Miscellaneous articles and equipment to be used in connection with erection of machinery, etc., contracts with numerous companies, the total amounting to about	14,000.00
Total		$13,993,589.08

machines used in excavating and handling dirt and rock in the dry.

Wet Excavation.—The plant for wet excavation included some 26 dredges, with scows, tugs, drill boats, house boats, rock breakers, floating pile drivers and cranes, etc. Seven of the dredges were renovated French machines and the remainder were built by the Maryland Steel Co., Sparrows Point, Md.; the Featherstone Foundry & Machine Co., New York City; the Atlantic, Gulf & Pacific Co., New York; the Bucyrus Co., South Milwaukee, Wis.; the Ellicott Machine Co., Baltimore, Md., and Wm. Simons & Co., Ltd., Renfrew, Scotland. These same concerns had contracts for barges and other floating equipment, and tugs, barges and service boats were purchased in the United States from various shipping concerns. The rock breaker used on the Pacific Sea Level Section was furnished by Lobnitz & Co., Renfrew, Scotland.

TABLE XLIII.

	To June, 30, 1909.	Fiscal year 1910.	Fiscal year 1911.	Total cost of plant.
Atlantic Division—				
Dry excavation plant	$ 196,507.74	$ 9,763.99	$ 285.45	$ 206,557.18
Dredging excavation plant	1,711,184.35	306,187.66	128,398.12	2,145,770.13
Gatun dam and spillway plant	826,794.83	117,141.54	576,839.19	1,520,775.56
Gatun locks plant	1,057,054.39	947,060.82	405,445.17	2,409,560.38
Gatun power plant (permanent)	194,462.88	....	194,462.88	....
Porto Bello rock plant	996,834.43	56,480.22	89,133.60	1,142,446.25
Nombre de Dios plant	235,973.20	160,088.80	26,172.56	442,234.56
Transportation plant	1,150,882.08	363,509.48	149,477.84	1,663,869.40
Colon breakwater plant	326,969.71	122,210.00	272,787.89	721,967.60
Total	$6,696,663.61	$2,082,442.51	$1,454,076.94	$10,233,183.06
Central Division—				
Dry excavation plant	$10,589,005.30	$101,412.26	$207,892.24	$10,279,700.80
Masonry plant	....	....	8,758.00	8,758.00
Total	$10,589,005.30	$101,412.26	$199,134.24	$10,288,458.80
Pacific Division—				
Dry excavation plant	$ 282,510.01	$ 19,999.47	$ 58,824.13	$ 361,333.61
Dredging excavation plant	2,244,385.70	597,832.22	145,224.79	2,987,442.71
Pedro Miguel locks and dams plant	323,551.00	479,830.19	32,291.70	835,672.89
Miraflores locks and dams plant	465,869.06	460,729.49	318,808.63	1,245,407.18
Ancon rock plant	327,914.15	441,043.87	47,368.51	816,326.53
Chame sand plant	162,105.52	133,352.07	19,910.77	315,368.36
Miraflores power plant	218,931.91	274,693.99	3,902.33	497,531.23
Total	$4,025,270.35	$2,407,481.30	$626,330.86	$7,059,082.51
Grand total	$21,310,939.26	$4,388,511.55	$1,881,273.56	$27,580,724.37

Concrete Plant.—A total of some 5,000,000 bbls. of cement was consumed in constructing the locks and other concrete masonry. The bulk of this cement was furnished by the Atlas Portland Cement Co., but some quantities of cement were bought in the open market and was of various other well known brands. The great bulk of the concrete work was concentrated at the locks and here permanent mixing and handling plants were installed. The unloading and lock cableways at Gatun were furnished by the Lidgerwood Manufacturing Co., New York. The concrete handling cranes at Pedro Miguel and Miraflores were built by the Wellman-Seaver-Morgan Co., of Cleveland, O. The Atlas Car & Foundry Co., Cleveland, furnished the electric railway for handling concrete materials at Gatun. All permanent mixing plants were equipped with cube mixers furnished by the Municipal Engineering & Contracting Co., Chicago. This company also furnished portable mixers for miscellaneous work and others were furnished by the Ransome Concrete Machinery Co. of Dunellen, N. J. Steel forms for the lock work were furnished by the Baltimore Bridge Co., Baltimore, Md., and the Blaw Steel Construction Co., Pittsburgh, Pa. The rock crushing plants were furnished by Allis-Chalmers Mfg. Co., Chicago, and the Power & Mining Machinery Co., New York.

Lock Machinery.—The lock gates and the machinery for operating the locks was erected by the several contractors named in the succeeding section and these firms used their own construction plant. Practically all handling of gate parts and operating machinery units was done with locomotive cranes, furnished by the Industrial Iron Works of Bay City, Mich., and these cranes were also used extensively in other sections of the canal work.

General Plant.—Besides special plant furnished for specific work there were large amounts of general plant and supplies. A schedule of those from whom such plant and supplies were bought would make a fairly complete list of American manufacturers of contractors' machinery and construction materials. Among the prominent firms were: American Steel & Wire Co., Chicago; Union Oil Co., San Francisco; Western Wheel Scraper Co., Aurora, Ill.; Pennsylvania Steel Co., Steelton, Pa.; American Car & Foundry Co., New York; Railway Spring & Wheel Co., New York; A. S. Cameron Steam Pump Works, New York; Otis Elevator Co., New York; B. F. Sturtevant Co., Boston; D'Olier Engine & Construction Co., Philadelphia; The Blake & Noel Co., New York; Alberger Pump & Condenser Co., New York; Builders' Iron Foundry, Providence, R. I.; Deane Steam Pump Co., Holyoke, Mass.; Maine Electric Co., Portland, Me.; Bethlehem Steel Co., Bethlehem, Pa.; H. R. Worthington Co., New York; American Bridge Co., New York; Buffalo Forge Co., Buffalo, N. Y.; American Bitumastic Enamel Co., New York; Platt Iron Works, Dayton, O.; W. N. Best & Co., New York; Yale & Towne, New York; Blackall & Baldwin Co., New York; Chicago Railway Construction & Supply Co., Chicago; American Hoist & Derrick Co., St. Paul, Minn.; Terry & Tench, New York; Swedish-American Electric Co., Chicago; Jones & Laughlin, Pittsburgh, Pa.; Carnegie Steel Co., Pittsburgh, Pa.; Cleveland Crane & Engine Co., Cleveland; Buckeye Engine Co., Salem, O.; Kittoe Boiler & Tank Co., Canton, O.; Chisholm & Moore Mfg. Co., Cleveland, O.; John A. Roebling's Sons Co., Trenton, N. J.; Hayward Co., New York; W. & L. E. Gurley, Troy, N. Y.; Clip Bar Mfg. Co., Philadelphia, Pa.; Broderick & Bascom Rope Co., St. Louis, Mo.; Concrete Steel Co., New York; Russell Grader Co., Paragon Mfg. Co.

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Engineering and Contracting

Devoted to the Economics of Civil Engineering Design
and to Methods and Cost of Construction

Volume XLI. CHICAGO, ILL., JANUARY 14, 1914. Number 2.

On Securing Expert Advice in Connection with Water Supply Projects.

In a recent editorial on "Excessive Self-Confidence" we named as one of the evidences of the existence of too great self-assurance on the part of engineers the comparatively slight demand for the services of consulting engineers. Particular mention was made of the small demand now existing for the services of consulting railway, highway and bridge engineers. In looking over the professional cards of engineers, as published in engineering journals, one is impressed by the large percentage of engineers making use of such cards who are specializing in water supply and associated branches of engineering. From this one might gather the impression that all is as it should be in connection with the securing of expert advice with reference to water supply projects. How much that impression is in error may be gathered from the following quotation from a letter which we received from a prominent consulting engineer on water supply projects in which he expressed his interest in and approval of the editorial to which we have referred. The quotation follows:

> As confirmatory of the position which you have taken, I might say that a very considerable portion of the work which comes into a consulting engineer's office—not only my own, but others with whom I am acquainted—is work which some too self-confident engineer or administrator has undertaken to do and got into trouble over. It goes without saying that it would have been easier in the first instance to have undertaken such matters than to straighten them out after they have been botched.
>
> Another large proportion of the work which comes into a consulting engineer's office comes with the assurance that the administrator or engineer who commits it to our care has been through similar experience in the past, and does not propose to make the same mistake again.
>
> A very small proportion of our work comes to us absolutely new and untouched by some such preliminary process, in which overconfidence has played too large a part.

While this engineer engages in many other lines than water supply engineering the fact that he does much work of this class warrants us in concluding, especially in the light of the final paragraph above quoted, that many of his water supply studies are undertaken after the home talent has unsuccessfully tried to handle them. We desire, in the present editorial, to call further attention to some of the aspects of specialist engineering advice in connection with water supply undertakings of every magnitude in cities of all sizes.

In this issue we publish an article on some of the aspects of water supply engineering in cities of from 300 to 3,000 population. We have previously stated in these columns that a high grade of engineering talent is essential to the proper working out of the water supply problem of cities of this size. This is especially true since only a small, fixed sum of money is available in a town of this size for use in securing a satisfactory water works system.

As Mr. Macartney points out in his article the officials of these small towns seldom realize the importance of securing competent advice before beginning the construction of a water supply system. The first cost of securing expert advice too frequently is considered prohibitive. This cost is near at hand and definite while the profit to a community resulting from this initial expenditure is comparatively far off and of indefinite amount. These considerations often prompt city officials to entrust the working out of the plan for the development of a water supply to some incompetent person who puts a low money value on his services.

Another view of the matter is presented by Mr. McCartney when he calls attention to the fact that there are many instances where poor advice and poor plans have eminated from supposedly competent engineers. In proportion as this condition becomes understood in cities in the vicinity of one which has been unfortunate in this particular, the chances of competent consultants securing profitable engagements in those cities is lessened. Some idea of the extent of this form of incompetency in the state of Washington may be inferred from Mr. Macartney's statement that to his certain knowledge 10 out of 60 alleged water works experts in the city of Spokane a year ago never had made a water works installation.

Mr. Macartney also calls attention to the opposition by engineers in private practice to the tendency of professors of engineering in state educational institutions to give expert advice on water supply matters to cities in the state. At the same time attention is called to the tendency on the part of city officials to place full confidence in the professional attainments and integrity of professors on account of the latter's association with state institutions. The engineering done by experts employed by supply houses is also mentioned and the too often biased nature of this type of advice is deprecated.

As corrective measures for the condition mentioned Mr. Macartney suggests either the licensing of engineers or the state approval of plans. The former is state approval of the engineer, the latter is state approval of his work, a vastly more desirable thing as we have many times pointed out.

In a journal of this character advice to the layman is of little value since it hits the mark only by accident. The foregoing observations on the practice of water supply engineering, insofar as they relate to the employment of consultants, however, prompts us to again call certain matters to the attention of the better class of consulting engineers, the engineering professor and the water works superintendent, all of whom are well represented among the regular readers of this journal.

We believe that consulting engineers for water supply will do well to work for the establishment of state control of water supply engineering where the practice of the profession has not as yet attained that stage of development. In this way only can the interests of communities be safeguarded, in general, from the blunders of the incompetent and the machinations of the biased and unscrupulous practitioner. This, of course, applies primarily to communities which are too small to employ regularly a competent water works man, either to install and operate an entirely new system or to extend and operate an old system.

The professor of water supply engineering is urged to limit his consulting practice in the field to such engagements as he may consider necessary to keep himself in touch with actual practice to the end that he may attain the highest efficiency as a teacher. An exception to this limitation of professional work will, of course, obtain where an institution, like the University of Wisconsin for example, gives advice to cities on water supply matters as a part of the university extension work. This work as carried on at the institution named is co-operative rather than competitive with the engineer in private practice.

Water works superintendents and engineers regularly employed by water departments and water companies are strongly advised to seek expert advice when important changes in or extensions to the system with which they are connected are planned. That this action is frequently taken we are well aware, but we feel that it should be taken in practically all cases. Many superintendents and department engineers are fully competent to plan any required improvement in the systems under their charge, but experience indicates the wisdom of having the plans approved by an expert before it is adopted to govern the actual construction, especially if it involves features outside the experience of the department. This applies with equal force to additional water supply undertakings of every magnitude. The wisdom of calling in expert advice on large projects was brought forcefully to the writer's attention a short time ago in examining the report of a board of consulting engineers on a project which will cost, finally, close to $15,000,000. On examining the plans prepared by the city's engineering department the consulting engineer found it advisable to make such a fundamental change as to recommend a gravity instead of a pressure conduit. The consulting engineers on this project also estimate that a sum of $500,000 or more may easily be wasted by failure to appreciate the elements in which economies may be practiced in working out the final detailed designs and recommended that an expert be placed in responsible charge of the final adjustments in the design.

We have occasion to read many reports of consulting engineers who are called in to pass upon previously prepared plans. Our observation has been that such reports almost invariably give full credit to the original designer for all meritorious work done by him. Coming from such an authoritative source such credit so increases the confidence of the home folks in their regular employe that they do not think the less of him for having asked that an expert be engaged to pass upon the merit of his work, even though certain changes may have tactfully been suggested.

Bridge Erection, with Comments on Methods to Be Used in Erecting the New Quebec Bridge.

Although there has been considerable improvement in recent years in the methods used in bridge erection, this part of the work still offers opportunities for reductions in the total costs of bridges. As conditions vary widely it is necessary to determine carefully the type of equipment best suited to the erection of each structure. Engineers are being brought to an appreciation of the fact that the erection features should be considered by them along with the design and fabrication problems. Often the type of design depends almost entirely on the method which must be used for erecting the structure. For the exceptional bridge the erection problems are generally given careful study, but for most bridges they are given little consideration until after the bridge is fabricated.

In many cases the erection of the bridge is the work of a man who has had nothing to do with the design, and who is not employed until after the bridge is practically ready to be erected. This man then must erect the structure with the apparatus which he has available, and as time is an essential element, he has little opportunity to make a careful study of the best methods to employ. There is little wonder that the erection often proves costly and results in delays. If the designer is consulted as to the methods to be used in the erection he often may be able to change some minor details which will greatly facilitate the erection of the structure and at the same time not increase its cost. It has been said that inferior erection methods mainly were responsible for the collapse of the old Quebec Bridge. Although the dead-load stresses were underestimated and the type of chord sections selected was open to criticism, the structure probably would have served its purpose if it had not fallen before it was completely erected.

In our issue of Jan. 7, 1914, we have recorded some of the erection features of the new Quebec Bridge. By examining the design and by talking with the engineer in charge of construction we were brought to realize that the erection of this bridge was considered the most important problem—even its design being of secondary importance. In designing this bridge the engineers constantly have had in mind a structure which readily could be erected and one which would not develop large secondary stresses during erection. It is not uncommon to make models of large structures, but in this case the engineers have caused to be made a large model, not only of the anchor and cantilever portions of the bridge, but of the erection traveler as well. In this manner many erection problems have been solved long in advance of the completion of the design.

The approach spans and anchor arms are to be erected on falsework, for which concrete piers have been constructed. As the bridge has a clearance of 150 ft. above high water, and as the water is very deep (about 190 ft.), it was impossible to use falsework for the main channel span. For erecting the anchor and cantilever arms two large and elaborate erection travelers are to be built. The cranes and hoists for these travelers are to be operated electrically, as is also the movement of the traveler. Most of the operations are controlled from a central switchboard located on the traveler. The suspended span is to be erected on barges, using one of the travelers, and floated into position. The care with which the erection problems have been worked out by the engineers of the St. Lawrence Bridge Co. should act as an incentive to engineers in this country to give increased attention to methods of bridge erection.

Credit for Conceiving the Plan of a Dam at Gatun for a Panama Canal.

Subsequent events have proved that it was the part of wisdom to have chosen a lock canal instead of a sea level canal at Panama. Of the lock canal plans suggested for Panama the one chosen providing for a summit level from Gatun to Pedro Miguel, a distance of 31 miles, was by all odds the most meritorious. Credit for the conception and adoption of this plan is divided and engineers will therefore read with interest the following testimony from Mr. Isham Randolph, one of the members of the International Board of Consulting Engineers who in 1906 reported in favor of a lock canal and who presented the lock canal plan which has virtually been followed in building the waterway now approaching completion:

> Sir: I have just read with great interest your editorial in the issue of Jan. 7 and naturally feel gratified by the credit extended to the minority members of the International Board of Consulting Engineers. What I am about to say is free of any intention to minimize the credit due to John F. Stevens for his great work in paving the way for the accomplishments afterwards brought about by the United States Engineer Corps. No one who knows the facts would ever minimize the credit which is due to him but I think you are mistaken in attributing primary credit to him for the Gatun Lake feature of the Panama Canal.
>
> You must go further back than that. In the report of the Board of Consulting Engineers of the Panama Canal for 1906, on page 279, Appendix No. 1, you will find the discussion of the Gatun dam by C. D. Ward, member American Society of Civil Engineers, which is a reprint from his paper on this subject which appears in the transactions of the American Society of Civil Engineers, Vol. 53, page 37, 1904, and from the references I conclude that Mr. Ashbel Welch, past president of this society, discussed it as far back as 1880, as the reference is to Vol. 9, page 148. In our deliberations, my recollection is that Mr. Frederick P. Stearns first brought forward the idea presented in Mr. Ward's paper and advocated it with such zeal that we of the minority were converted to his way of thinking.
>
> There is one other statement in your editorial which is not in accord with the facts as I understand them. Speaking from memory without reviewing our report in any way, the lock foundations between Sosa and Ancon were found to be satisfactory and the dam across the Rio Grande was subject to less criticism than the dam across the Chagres at Gatun. My understanding has been that the locations which were selected for the locks were rightly regarded as unfit by the army engineers on account of exposure to gun fire from any hostile fleet; and to overcome this, the locks were moved back to Miraflores and the large lake on the Pacific side abandoned.
>
> As to the matter of slides, although we dipped into the hillside "as far as human eye could see," the results have proven that the power of the human eye in such cases is woefully defective.
>
> The historical and engineering data presented in this number are of very great value and will be appreciated as much by those who are anxious to know the facts in regard to the Panama canal as by the professional public.

Mr. Randolph as a member of the International Board of Consulting Engineers appointed to recommend the type of canal to be constructed at Panama speaks with authority of the considerations which influenced that board in making its decisions. With only the bare printed record to guide one it is difficult always to apportion credit fairly to those who deserve credit for notable performance. We should perhaps more properly have credited Mr. Stevens largely and not primarily with having aided in the notable decision of the minority members of the International Board to recommend a dam at Gatun and a great artificial lake waterway from Gatun to Bas Obispo. Gatun lake and dam were not, as our article states and as Mr. Randolph's letter reminds us, first conceived by Mr. Stevens. This credit probably belongs to a Frenchman, Godin de Lepinay, one of De Lessep's "international commission" of 1879, and Mr. Welsh in 1880 and Mr. Ward in 1901 recreated the thought. Gatun lake and dam were merely a conception until Mr. Stevens made the preliminary studies at Gatun which convinced him and the commission, of which he was chief engineer, not merely that in these structures there was a notable idea but that these structures were engineering and constructional practicabilities. In his book, the "Panama Gateway," Mr. Joseph B. Bishop, secretary Isthmian Canal Commission, says:

> Mr. Stevens, who succeeded him (Mr. John F. Wallace—Eds.) in July, 1905, was impressed on his first careful study of the field by the superior advantages of the Gatun site, and had such additional borings made as convinced him of its suitableness. He recommended it earnestly both to the canal commission and to the International Board of Consulting Engineers when that body visited the isthmus in 1905. Its adoption by the minority members of that board in the plan of the canal recommended by them and approved subsequently by Congress was due largely to his advocacy.

Obviously Mr. Randolph's testimony of the deliberations of the International Board should be of greater authority than this quotation; he generously gives chief credit to Mr. F. P. Stearns for converting the minority members to the Gatun lake and dam plan.

Mr. Randolph's statements that the Sosa locks site was abandoned for a site farther inland at Miraflores for military reasons chiefly, is, we imagine, more nearly the real reason for the change, than were unstable foundations at Sosa, but ostensibly the change of location was made because of unreliable foundations. In his report for 1908, Col. Goethals says:

> The project proposed by the minority members of the board of consulting engineers for the lock type of canal provided for a flight of three locks at Gatun, a flight of two locks at La Boca, and one at Pedro Miguel. The locks at La Boca were placed on the west side of Sosa Hill, and were to overcome the difference of the level between the ocean and a lake formed by the Sosa-Corozal and the Sosa-San Juan dams. Steps were taken to construct the former of these dams in accordance with the proposed plan, and trestles were built along the toes from which to dump material from Culebra cut. The trestles failed after the dumping from them began, and the material overlying the rock moved laterally, carrying the superimposed mass with it. In places this lateral motion continued for two weeks after dumping had stopped. The ground on either side of, and for some distance from, the dump was forced up, forming mounds of mud, the crests of which gradually approached the level of the dump proper. After an equilibrium was established between the dump and the adjacent mounds the hump or wave would again move out when the track was shifted toward it, accompanied by a sudden vertical settling of the track of 6 to 10 ft. when loaded trains were applied. Dumping from trestles of less height secured no better results.
>
> When the difficulty developed, investigations by borings and test pits were made to determine the character of material overlying the rock. It was found to be, for the greater part, an unctuous blue clay without grit, possessing very little supporting power, instead of a stiff clay, as indicated on the profiles. The depth of rock below the surface varied from 10 to 70 ft., and in order to build the dams more money and time would be required than had been estimated.
>
> When this became evident, a careful examination of the canal route from Pedro Miguel to the Pacific was undertaken, by wash and diamond drill borings and test pits, to ascertain if a more suitable place for the locks and dams could be found. In making these examinations care was taken to secure samples of the materials to be encountered at the various depths, as well as cores of the rock for depths of 25 ft. below the foundation level of the lock walls.
>
> A study of the data thus obtained led to the conclusion that one lock at Pedro Miguel and two at Miraflores offered the most economical and desirable solution. The advantages of this plan over the then existing project were that dams of lower height, less length, and resting on rock comparatively near the surface could be more easily constructed and could be completed at an earlier date; and finally that the locks in this location would be protected against all possibility of distant bombardment and would be less exposed to gunboat or torpedo boat attack. As a consequence, the commission recommended a change in the project, which received the approval of the President on December 19, 1907.

Few engineers, we imagine, who have read the evidence in detail will be inclined to attach any considerable blame to the engineers who aided in planning the Panama Canal for not foreseeing the slides at Culebra. It is very doubtful whether they could have been foretold by any character of investigation less inclusive than actual excavation of the cut.

Buildings

Data and Cost of Driving Piling for a Five-Story Bank and Office Building at Chickasha, Okla.

Contributed by F. O. Kirby, Superintendent, Columbia, Mo.

In excavating for a foundation for the boiler-room of a five-story office building at Chickasha, Okla., soil conditions were such that it became advisable to sink a test pit for investigation. The building is five stories in height and 100x100 ft. in plan. It is arranged for a bank and stores on the first floor, and for offices from the second to the fifth floors. All columns and floors are of reinforced concrete, the Kahn system, "Floretile" being used for the floors.

Only that part of the basement which is to be used for the storage of coal and for the boiler-room and the storage vault was excavated, and the footings were carried to such a depth that they were in or were near soil containing quicksand. The excavated portion is shown by the part plan, Fig. 1. All other footings were carried to a depth of about 5 ft. below the floor line. Figure 1 also shows the location of the 164 piles for the footings. The average allowable load per pile, given for each footing, was computed from the formula, $P = \frac{2\,WH}{s+1}$; where P = allowable load, in tons; W = weight of hammer, in tons = 0.6; H = fall of hammer, in feet; and s = penetration under last blow, in inches.

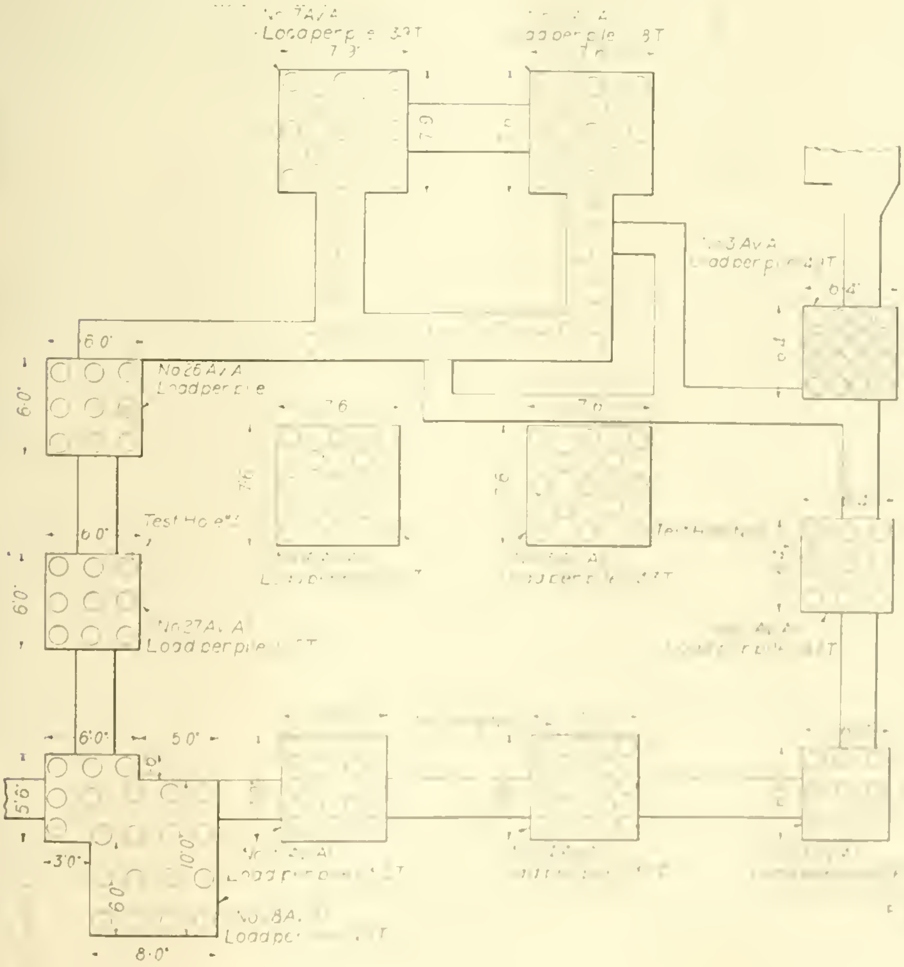


Fig. 1. Plan of Excavated Portion of Building at Chickasha, Okla. Showing Location of Pile Footings and Test Pits.

GENERAL SOIL CONDITIONS.

At one time there was a slough where this building now stands; and as the town grew and the street was built up, the ground was filled in. In general the soil on top of the ground to a depth of about 10 ft. is a red sand clay, with strata of gumbo and hard-pan every few feet. Below a depth of 10 ft. the sand in the soil increases. The high-water mark, during the wet season is about 6 ft. below the street level, and the low point, as shown in Fig. 2, is about 17 ft. 6 ins. below this level. The work was done in the fall of 1912, a very dry season, and the water was at a fair average of the low-water level. Below this level for about 13 ft. 6 ins. was muck and quicksand; the next 2 ft. showed muck, with lumps of hard-pan; and the following 7 ft. graded into hard-pan. At this point, 40 ft. below the street level, the pipe went into a pocket of quicksand for about 10 ft. At a depth of 50 ft. it changed from quicksand to a fine sand, which became coarser farther down. We did not drill through the rock to get its thickness. The same strata were found in each of the other two test pits. The first test pit was taken in the center of the basement, and the other two were taken at each side of it (see Fig. 1).

TEST PITS.

Pits Nos. 2 and 3 were made by a post auger, using pipe to lengthen the handle. For pit No. 1 we started by driving a galvanized iron pipe in sections by leverage. At a depth of 25 ft. this pipe closed up so that we could not get a 3-in. auger through it nor could we use a sand bucket to clean out the pipe. We then rigged up a driver by using a set of leads with a railroad tie for a hammer. Laborers were employed to lift and drop the hammer, to drive a 1-in. pipe in 10-ft. sections, and to take out the soil with an auger and sand bucket.

The cost of the three test pits shown in Fig. 2, including materials, is given in Table 1. This cost is based on 1912 prices for materials, the local union scale of 45 cts. an hour for carpenters, 20 to 25 cts. for laborers, and 62½ cts. for the foreman. The lumber in the derrick is not included in the cost, as it was used in the building.

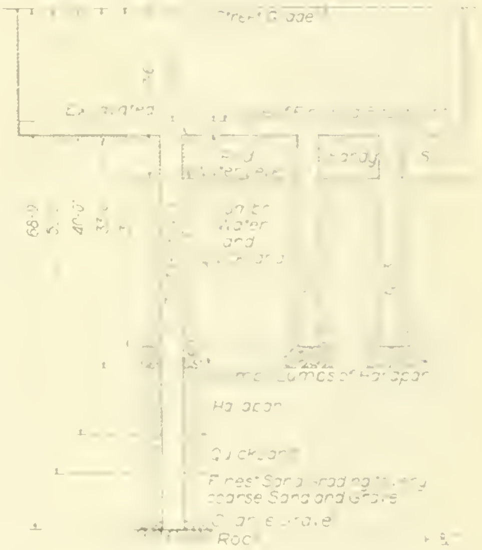


Fig. 2. Depth of Test Pits and Character of Sub-Soil.

TABLE 1 — COST OF LABOR AND MATERIALS FOR THREE TEST PITS SHOWN IN FIG. 2.

Item.	Cost.
Driving test holes and removing wrought iron pipe, labor only	$ 31.45
Carpenters, building derrick	7.20
Carpenter, rigging	3.60
Post auger	2.00
15 ft. of 1-in. black pipe	1.05
15 ft. of 1-in. No. 22 well casing	2.70
1 sand bucket	4.75
12 ft. of 1-in. black pipe	0.78
27 ft. of 9-in. galvanized iron casing	[illegible]
2½-in. sand bucket	3.75
62 ft. of 3½-in. black pipe	28.52
2 couplings	0.80
2 caps	0.90
1 B. coupling	0.10
4 cuts and threading	1.60
13½ ft. of ¾-in. black pipe	0.78
2 ¾-in. couplings	0.15
Cutting and threading	0.25
Ring in pipe	0.15
4 girders for derrick	[illegible]
2 braces	0.50
Clamp to pull pipe	[illegible]
25 ft. of 9-in. casing	[illegible]
Total cost of test holes	[illegible]

The log of the soil was given to the architects, and they were requested to drive piling and to choose the spacing and length of the piles. Owing to the fact that the strata contained hard pan at a depth of 33 to 40 ft., that the strata graded from quicksand to hard pan, and that there was a pocket of quicksand under this, it was thought best to use 30-ft. piling. The spacing of the piles was assumed, and it was also assumed that the safe load would average about ten tons per pile, using the formula $2\,WH/s+1$. The average load per footing was 11 tons. A reading was taken on the footings outside of this basement to see if any settlement was caused by the driving of the piles. With the exception of the two footings next to the basement none showed a settlement; these two settled about ⅛ in.

CONSTRUCTION AND COST OF PILE-DRIVER.

As this work was unexpected, an agreement was made with the contractor to do the work on a basis of cost plus 10 per cent, the owner to furnish all materials and tools. In the preliminary estimate, no allowance was made for an extra depth of the footings, caving of banks, etc. The Rock Island Railroad Co. permitted the use of a hammer and cap, but we furnished the leads The weight of the hammer was 1,200 lbs. Figure 3 shows the construction of the pile-driver and Table II gives its itemized cost

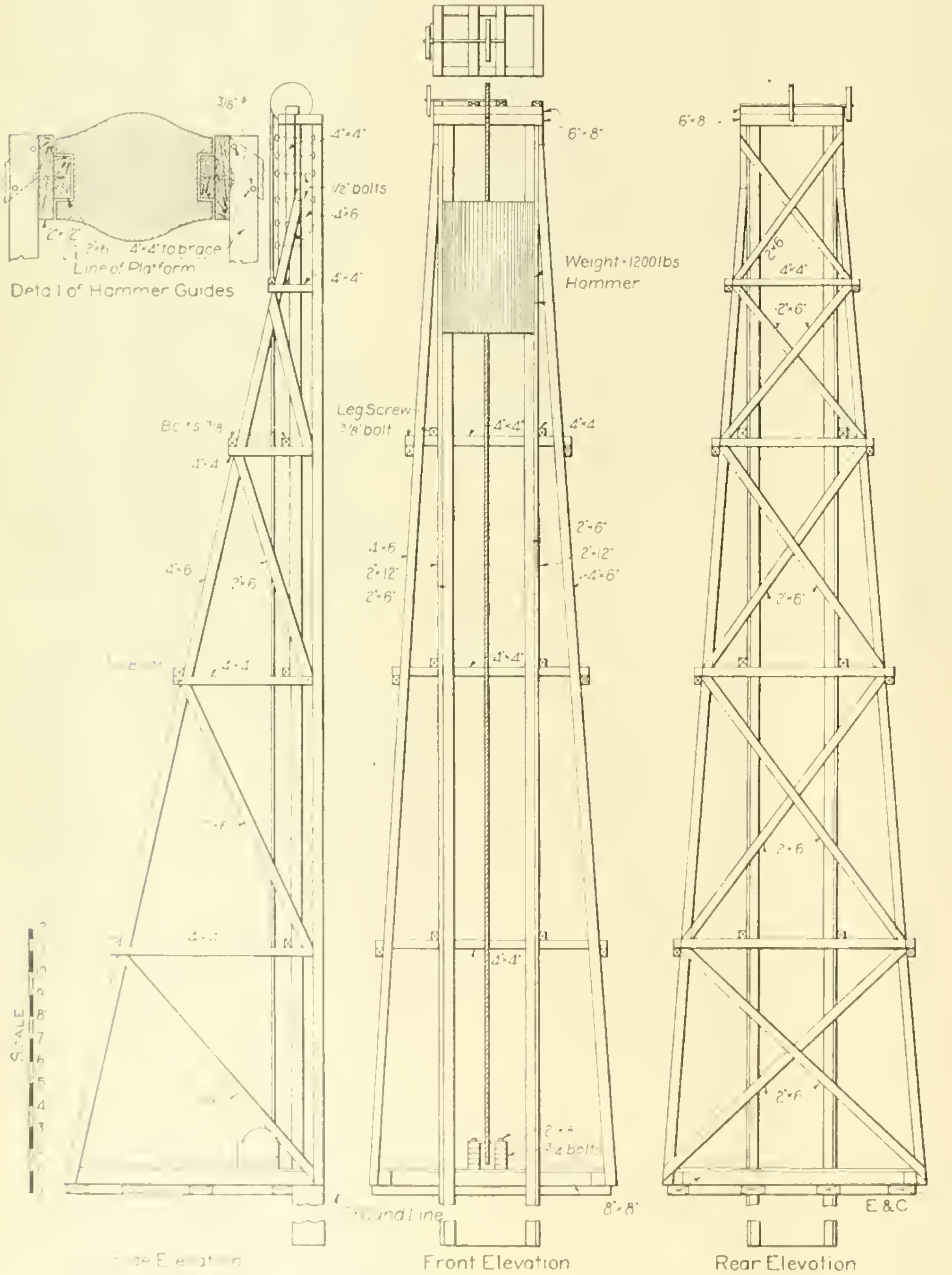


Fig. 3. Pile-Driver Constructed for This Work—Drawing Shows Type of Construction and Size of Members.

TABLE II—ITEMIZED BILL OF MATERIAL AND COST OF PILE DRIVER.

Item.	Cost.
888 ft. 4x4 ins.	$ 24.65
16 2x6 ins. x 12 ft.	7.15
1 4x4 ins. x 8 ft., 2 4x4 ins. x 14 ft., 1 6x8 ins. x 10 ft., 8 4x6 ins. x 16 ft.	10.15
5 8x8 ins. x 10 ft.	8.55
11 4x4 in[illegible]	6.95
2 8x8 ins[illegible]	5.59
2 6x8 in[illegible]	4.70
6 2x6 ins [illegible]	7.50
1 4x6 in[illegible]	1.20
30 2x10 i[illegible] x 16 [illegible]	23.60
3 6x8 ins [illegible]	7.10
3 4x4 in[illegible] x 10 ft. [illegible]x4 ins. x 16 ft., 16 2x12 i[illegible] x 16 ft., [illegible] 4x6 ins. x 16 ft.	19.75
10 2x1[illegible] x 16 ft., 20 4x4 ins. x 12 ft.	14.40
15 [illegible] x 1[illegible], 10 2x6 ins. x 16 ft., 10 4x4 ins. x 16 ft.	10.0[illegible]
20 2x6 ins x 16 ft.	8.85
Total lumber bill charged to driver	$ 160.10
Miscellaneous items, hardware, etc.:	
2 braces, ½x1½, x 18 ins., rope shield	$1.50
4 toggle irons, 5/16x2x25 ins.	2.50
2 braces ½x1½x42 ins.	1.25
Sleeve on clevis pin for driver	0.25
6 eye bolts for driver	0.30
2 toggle irons	1.10
2 staples and 2 pieces of pipe	0.25
2 staples and brace	0.40
Filing three saws, and guide for rope	1.80
4 band plates 2½x22 ins.	1.30
Key for crank and extension	0.75
26 ⅝x6-in. bolts	2.07
72 ½x6-in. leg screws	2.70
17 ⅜x7-in., 12 ⅞x7-in., 19 ⅜x8-in. bolts	3.06
5 ¾-in. washers, 2 ⅞x15-in. bolts, 24 ½ x7-in. bolts	0.86
4 ½x12-in., 12 ¾x7-in., 12 ½x7-in. bolts, 36 ½-in. washers	0.89
8 ½x20-in. bolts, 12 washers, 6 sledge handles	3.05
1 1x10-in. bit, 4 ⅝x18-in. bolts, 100 ⅝ x5½ leg screws	3.40
100 ½x3-in. leg screws, 12 ½-in. washers, 6 ½-in. bolts	3.25
12 ⅜x9-in., 4 ½x10-in., 4 ½x20-in., 12 ½x4-in. bolts	0.95
13 W. bolts, 8 ½x1-in. bolts	0.80
310 ft. of 1½-in. rope, phone, express	37.95
150 ft. of 1-in. rope	6.90
3 ⅝x37-in., 24 ½x7-in. bolts, 2 washers	1.52
7 dollys, wire, express	42.25
1 2½x9-in. bolt, ½-in. washer, 12 ¾x 16-in. bolts	1.20
125 ft. of 1½-in. rope, phone, express	15.80
12 ⅜x3½-in. bolts, washers	0.35
1 nut, 2 bolts, 14 1-in. screws, axle grease	0.61
6 ⅝x24-in. rods, 5 ¾x44½-in. rods, 10 cast washers	3.95
2 large sheaves, 2 shafts, 4 bearings, 1 angle, drilling	19.40
2 ¾x24-in. bolts, 4 ½x34-in. rods, 8 nuts, 8 washers	1.70
11 plates ½x3 ins. and drilling, 2 rope guides	2.40
500 ft. of 1¼-in. rope	19.01
Total hardware and miscellaneous charged to driver	$ 184.57
Labor cost of building pile driver:	
Carpenter labor	$ 125.95
Common labor	33.75
Foreman	9.80
Total cost of labor	$ 169.50
Total cost of pile driver	514.17

COST OF PILING, HANDLING AND DRIVING.

Item	Cost
Cost of piling and handling:	
8 6-in. rings, 2 7-in. rings, 4 large pile rings	$ 19.10
165 miscellaneous rings	93.00
6 cant-hook handles	3.00
1 No. 0 counter	2.30
Drayage on piling	82.50
165 L. L. Y. P. peeled, 30-ft. piling	1,574.00
Total cost of piling and handling	$1,773.90
Driving piling, day crew:	
Foreman, at 62½ cts. per hour	$ 59.50
Carpenters, at 45 cts. per hour	168.45
Common laborers, at 20 to 25 cts. per hour	316.05
Engineer, at 45 cts. per hour	59.85
Total on day work	$ 603.85
Driving piling, night crew:	
Electric lights	$ 21.40
Foreman, at 75 cts. per hour	13.00
Carpenters, at 50 cts. per hour	28.30
Laborers, at 25 cts. per hour	208.85
Engineer, at 75 cts. per hour	13.00
Total on night work	$ 284.55
Total cost of labor, day and night crews	888.40

COST OF MISCELLANEOUS ITEMS.

Item	Cost
Miscellaneous items charged to the work as a whole:	
Coal	$ 70.23
Telegrams	7.46
Capitol Steel & Iron Co., extra steel in footings	279.09
Texas Co., oils	4.95
Miscl. freight bills, drayage	16.50
Total miscellaneous items	$ 378.23

COST OF SHEET PILING AND DRIVING.

Item	Cost
Sheet piling, mill work and handling:	
174 2x6 ins. x 16 ft., 226 2x6 ins. x 18 ft., 100 2x6 ins. x 14 ft.	$ 222.80
50 2x6 ins. x 24 ft., 43 4x4 ins. x 16 ft., 17 4x4 ins. x 18 ft.	71.30
75 2x8 ins. x 16 ft., 100 2x4 ins. x 14 ft., 170 1x6 ins. x 11 ft., 88 1x6 ins. x 16 ft.	110.05
10 6-in. tops for mauls	1.35
Working 400 2x6-in. x 16-ft. pieces at planing mill	39.00
6,400 ft. of B. stop	103.20
Drayage on lumber to mill	5.10
Total lumber and mill work	$ 552.80
Cost of driving sheet piling:	
Foreman	$ 15.00
Carpenters	80.75
Laborers	50.95
Engineers, pumping water	14.85
	$ 161.55

EXCAVATION, CUTTING OFF PILES AND SHORING.

Item	Cost
Excavating walls for driver.	
Foreman	$ 22.00
Laborers	170.05
Total	$ 192.05
Excavating around piling below water line:	
Foreman	$ 25.50
Laborers	313.50
Engineer, pumping water	21.60
4 sharp shooters	4.00
2 14-in. spades	2.00
Total	$ 366.60
Cutting off piling, with one-man saw:	
Carpenter	$ 56.90
Engineer, pumping water	19.80
One-man saw	2.00
Total	$ 78.70
Shoring walls to prevent caving:	
Foreman	$ 11.00
Carpenters	19.80
Laborers	76.50
850 ft. B. M. lumber, furnished by contractor	22.95
	$130.25
Excavating on account of caving of banks	$60.48
Back-filling, labor	146.75

COST OF HOISTING ENGINE REPAIRS.

The contractor made no charge for the use

∠ aRC = 90°

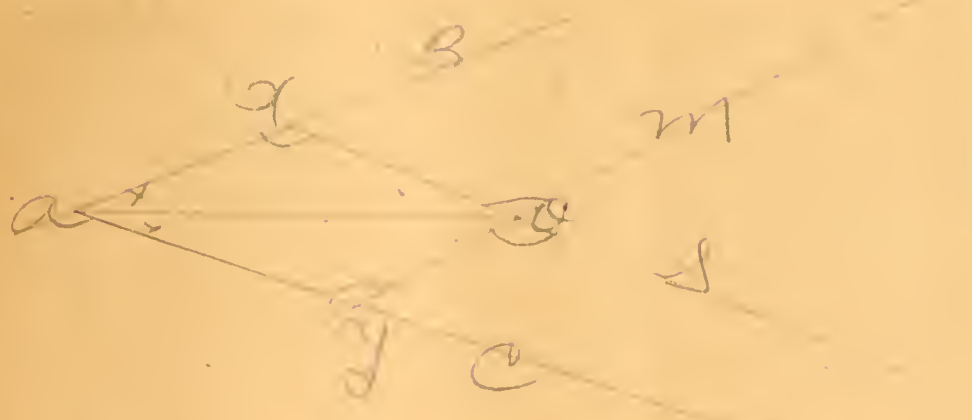
∠ aEB = 90°

∠ aRE = ∠ aEB.

∠ BaE = ∠ EBR 90

∠ BaE + ∠ EBR = 180°

H.P.



Hyp. ∠ BaO = ∠ Oac

xO // ac

yO // aB

Prove xO = Oy

proof ~~∠ BaO~~

x O // ay and yO // ax [illegible]

∠ ayO = ∠ yOS

yO // aB [illegible]

Labor	$1,333.70
Piling	1,773.90
Sheet piling and labor	714.35
Total cost	$3,821.95

Cost per footing, total = $\frac{\$3,821.95}{13}$ = $293.99

Cost per pile, labor only = $\frac{\$1,333.70}{}$ = [illegible].13.

harmony in their business relations will result.

The writer believes most positively that an engineer and a contractor are both necessary. That no matter how great the ability in every direction of any one person, and in spite of unlimited help and financial resources, the owner's interests in the long run can better be served by two persons—the engineer and the contractor—working together than by one

rforming all these duties. Holding e views, the writer wishes to make that no criticism of engineers and is either made or intended in what There are some plain statements however, made to illustrate the con- point of view.

mpany with which the writer is con- is for nearly 20 years specialized in d concrete. In the past it has ex- ork of every kind in which Portland s largely used, but during the last s nearly every class of cement work dropped except building construc- e goal the company has sought by ializing and concentrating is to be erect a better building, in quicker for less money, than any other con- is part of the United States. It will n what follows that the day of the whose only written record is a ne book and the stubs of a check passed, and that somewhat intricate and elaborate organization have place. It is hoped to prove that in ne contractor's problem of furnish- efficient and economical service the complished justify the methods em-

ing described in this paper is from al experience of the writer and of ny. In order to show as clearly e the activities, duties and work ntractor it has been found necessary cases to describe a specific exper- generally only the fundamental pplicable to the execution of any iscussed

bject will be discussed under the headings: Human Element; The Preliminary Work; Analysis of the election and Purchase of Materials; nt of Construction; The Plant; on of Job Force; Planning, Rout- ost Accounting; and Weather Con-

HUMAN ELEMENT

e well to consider the human ele- ne problems of the contractor, an hich is sometimes paramount to the cution of a contract.

arning that there is a prospective ber of the company investigates its and location, to determine whether n the company's specialized work, er it is desirable. He then meets if possible, to interest him, and his temperament and business

er.—As sometimes the owner takes interest in the execution of the ugh he deals through his engineer t, it is of considerable importance s temperament and point of view. mony will thus be obtained. The ill do well to bring both parties rly and often. There is the owner not been seen at the time of final and the one who is in evidence every one who has a technical training s, and the one who does not know nstruction, but thinks he does, and ly mixes into the work commonly ed as the engineer's. If the engineer k-kneed," with the last named type of expense and misunderstanding are al ertain to result, primarily because no an serve two masters and secondarily e an ignorant master, the owner, is satisfied. If there is a very strong, fair- ed engineer who commands the respect is client, there is little trouble with any of owner.

he worst combination is a fussy owner l an engineer who has obtained his commission by hard chasing, instead of through the owner's going to him. He is the hardest to control and makes trouble for the contractor that affects the cost of the work. The contractor, therefore, seeks to learn the probable conditions before submitting a bid.

The Engineer or Architect.—Then it is necessary to study the engineer or architect and consider his method of handling and supervising work. The methods of some engineers are radically different from those of others; this has a marked bearing upon the cost of the work and sometimes on its desirability to the contractor. The contractor must determine what sort of treatment he will receive. Nothing adds to the price of a bid like uncertainty, whether it be in the temperament of engineer and owner or in unknown construction problems.

In one case where a number of local firms were invited to bid on a certain large building for an architect who was well known always to rule in favor of his client, irrespective of the merits of the case, the successful bidder, after figuring full measure on all quantities, added this item: "Humor architect"; the item was 10 per cent of the contract. It is safe to presume that all other competitors placed this contingency at a higher figure. Such a man could not get a reasonable bid from any one who knew him, and he certainly could not expect to let work advantageously to guileless strangers. An engineer in a distant city whose reputation has reached beyond the limits of his activities has a personal equation of 25 per cent plus. The best information available indicates that these men do not personally profit in any way improperly in this excess cost. Instead of being wise and just judges between the two contracting parties, they are simply over-zealous partisans. Fortunately this class of architect and engineer is very small and their influence on industry still smaller.

The largest class is that in which the engineer or architect intends always to be just, but on rare occasions allows his judgment to be influenced by the attitude of the owner; or, where two different solutions may be possible, for policy's sake or for hope of personal advancement, makes the second best decision. This increased expense which is entailed can never be foreseen, although it certainly exists and has to be paid for as a contingent item of cost. The reputation of the engineer who is always fair spreads far and wide, and he is sought out by many responsible contractors who desire to do his work. This results in work being done at the lowest legitimate cost by competent firms; the owner gets a square deal, continues himself, and also recommends his friends, to employ this engineer, who in the long run prospers more than he could by other methods. Meanwhile, he maintains his self-respect and is justly proud of work well done.

Contractors talk rather freely among themselves about their experiences and their opinion of those for whom they have worked, and perhaps oftener than is imagined are asked by owners whom they would recommend to design new work. They certainly exert an influence on the reputation and also on the business of the engineer.

The Inspector.—One of the great problems which the contractor has to face is that of the inspector who represents the engineer on the work. A thoroughly competent inspector is a great help to the contractor and very beneficial in the results obtained. An incompetent one is exactly the reverse. The writer recognizes the difficulties which the engineer labors under in selecting his inspector. The duties are not hard, and to the man with ability, ambition and push, the close confinement to long hours of attendance on the work with little to do except keep his eyes open is too irksome to be long endured, even at a good salary, and this desirable type soon moves to what he considers better employment. The most important thing to consider in the appointment of an inexperienced man in his temperament. He must have an even temper not easily ruffled, be considerate, yet firm, always diplomatic, and be capable of performing his duties acceptably to his employer.

The superintendent for the contractor is necessarily better informed and trained in construction, an executive who is constantly directing others and quick to resist for the good of the service insubordination or usurpation of the duties for which he is held to strict account by his employer. In other words, he is trained to be an autocrat. This type of temperament does not take kindly to being bossed by a younger man admittedly with less experience. Therefore, the inspector must have the above-named qualities and also must not consider it a personal affront when his orders are overruled. He will make some mistakes which the engineer will correct, and many more, alas, which his chief will never hear about, although they make unnecessary expense and may affect his chief's professional reputation. He is put in a trying position, left too much alone, and required to perform certain duties, but he must not go beyond bounds.

Friction is most frequently caused when the inspector assumes to perform duties which do not belong to him. The contractor has a perfect right to object to this, but out of the refusal grow many complications. Judging from results, the inspectors are not concisely instructed in their duties and they are not watched closely enough by their chiefs to see that they not only perform their duty but also do not attempt to perform more. The most common mistake of the inspector is to interfere in the conduct of the work by giving the workman orders direct. To permit this would destroy the morale and disorganize the job with far-reaching consequences.

The inspector appointed for political purposes is a thorn in the flesh. In one case where a granolithic sidewalk was being laid for a city, an incompetent inspector had a tar concrete specification as a guide. Another city inspector demanded a cash payment before he would pass a sidewalk pavement.

On a job for the state a transitman was appointed to inspect some granolithic paving. He was thoroughly conscientious and honest, but entirely ignorant of the work he had to inspect, and thoroughly suspicious of the contractor.

It may be asked why the contractor so seldom protests against the rulings and objections of incompetent inspectors and those who have an alleged grudge to work off, and appeals to their chief. Experience has proved that the firm which habitually does this would find it much more profitable to retire from business.

The Superintendent.—The very first step which a contractor must take in starting the execution of a job is the selecting of a general superintendent to handle the work. The type that is hired for a single job and discharged at its completion is not worth having. The really desirable superintendent is a development from experience, the one survival from many tried, and when once obtained a firm cannot afford to lose him. Some owners and engineers appreciate the value of personality so much that they have given the writer's company contracts under condition that a certain superintendent be put in charge.

Frederick W. Taylor, in his excellent paper entitled "Shop Management," specifies nine different qualifications which go to make up a well-rounded man, namely: "Brains; education, special or technical knowledge; manual dexterity or strength; tact; energy; grit; honesty; judgment or common-sense; and good health." He states that there are plenty of men to be found who embody three of these qualifications. Four make a higher-priced man. A man combining five is quite hard to get, and one combining six, seven or eight is almost impossible to obtain.

If a building superintendent is to be successful, he must combine at least seven of these qualities. He must have brains, special and technical knowledge of both direct contract and sub-contract work, tact, energy, honesty, judgment and good health. He must have a personality which drives to activity several hundred originally unorganized men who are without special interest in the company for which they work or in the result accomplished, and with such tact and judgment as to weld them into a harmonious working force, cheerful and self-respecting, with high morale, and ultimately with enthusiasm for the work in hand. He carries a care so great that he builds in full size, with permanent materials, the intricacies of design which trouble the engineer's drafting room to show clearly on paper; with an honor so fine that the company is ready to leave its reputation in his hands, to trust him with funds; and with special experience so trained that dangerous operations are carried on as a matter of routine, without worry to himself or the company, yet with a constant oversight of a thousand chances for accident or perhaps death which may occur to the men in his charge; with a forethought so great that he sees ahead and provides for the problems which are to come up perhaps months later; with a temper so good that he never loses self-control under the most provoking circumstances, and is able to take with the best of grace changes in his plans from the office, and to work in the close co-operation with the company which is so necessary to make it an effective contracting organization.

Such men have a temperament that responds quickly to criticism or praise. Praise comes sparingly, even when deserved, while criticism is freely meted out. Superintendents in the employ of the company have recalled to memory words of appreciation from an owner or an engineer long after the job has faded from the writer's memory, and he has seen a man's work improve in quality and cost purely through praise for some detail of the work which was ably handled or some difficulty which was ingeniously overcome. The company, including its superintendents, feels as much pride in the jobs it does as do the engineers who have designed these structures. The members of the executive force on any job which is sharply criticised will try to avoid criticism by refusing the slightest responsibility beyond what they believe to be clearly their own. A company sharply criticised by an engineer is likely to do precisely the same thing and will throw on to the engineer every bit of responsibility which it can possibly avoid. The attitude of the engineer in this respect is reflected in the execution of the work. One engineer may call attention to a mistake with a letter that is harsh and ends with a sting, which leaves the feeling of injustice and soreness in the recipient. Another, in calling attention to a similar matter, ends his letter with some expression like the following: "We appreciate your wish to make this work as satisfactory as possible, and recognize that this occurred through failure to understand my exact requirements." In response to such a letter as the first, the tendency is to do just as little as will satisfy the engineer and take your own time about it. But, as the writer knows from personal experience, in response to the second letter, you jump to correct the trouble cheerfully, quickly and without comment, and also sometimes do more than was asked for.

The engineers for whom we have done the best, cheapest and the most cheerful work have uniformly trusted us, have assumed that mistakes which we made were excusable, have co-operated with us to untangle difficulties which we have gotten into, and have been appreciative.

The problem in selecting the superintendent which the contractor must consider, is whether in the particular location he will be able to handle the difficulties which arise. There are superintendents who invariably command the complete confidence of the owner, no matter what his disposition may be, who always get along nicely with the engineer and his inspector, although they may be lacking in some other qualities which are desirable. Perhaps the owner and engineer desire an exceptionally fine appearance in the finished mill. One superintendent is especially good at this. Perhaps finish is of no great moment, and business ability is, on account of the job being isolated so that the superintendent is left alone for some days at a time.

Perhaps the local conditions may demand a great deal of tact by the superintendent in the handling of his labor. The following illustration shows what tact is required. When the writer's company executed its first contract in Buffalo, it was for a firm which had had considerable trouble with labor and it was anticipated that there would be a strike before the job was very far advanced. This firm had been marked by the local labor organizations as their natural prey, and these organizations were also prejudiced against outside firms coming into their territory. The carpenters there have a strong organization. During the early stages of this job, while there were but a few carpenters, the superintendent could give them considerable personal attention and things went smoothly, but as soon as work began on the second floor where they could not be so easily seen and the superintendent was too busy with other matters to watch them closely, unit costs began to climb day by day. The superintendent studied the situation to find the cause. By the time form work was starting on the third story he became convinced that the union steward of the job was to blame and was holding the men back from doing their best. The natural impulse would have been to discharge him immediately, but that would have made hard feeling with the union. The superintendent took this man aside, confided to him his troubles, and then made this man sub-foreman with entire charge of erecting forms for columns, which was the particular item which showed the highest cost. Immediately the costs came down, and on the fourth story were the lowest on the whole job. The result was saving a thoroughly first-class workman and keping in the good graces of the local organization. Some years have passed, and there has been no trouble whatsoever with the labor situation in Buffalo. Such tact and forbearance are frequently demanded of superintendents, and they are usually equal to the situation.

These various questions must be weighed and settled before a start is made. Experience has shown that after a job has once started with a given organization, a change in the superintendent is the cause of much disturbance to its satisfactory completion and economy in the handling of labor. It is of the most vital importance that this question be settled rightly once for all.

THE CONTRACT.

The type of contract is manifestly of considerable importance. It is the firm opinion of the writer that the usual type of lump sum contract obtained in competitive bidding does not give an owner the best results. With competitive bidding open to all, the lowest bidder is liable to be one who does not use the best of methods, is looking for all sorts of short cuts, and is frequently one with a limited amount of experience and capital. It often has been noted that those new in the reinforced concrete field underestimate the difficulty and cost and do work badly at first, no matter how experienced in other lines. The lowest bidder is squeezed to a figure where it is known he cannot make much, if anything, and there is every incentive to save and to slight. This condition of necessity encourages mutual suspicion and antagonism before the start; therefore, the best results are impossible. No amount of careful inspection can make an inexperienced or incompetent contractor turn out thoroughly first-class work.

On the other hand, under the cost-plus-fixed-sum-for-profit type of contract, the interests of the owner, builder and engineer are one. Work obtained in this way is certain to be much better for the sAme cost; even if the cost is slightly greater, the owner has the assurance that he is getting much more for his money. The majority of manufacturers whom the writer knows look upon a mill building as a tool to be used in a manufacturing process, rather than as real estate, and want to get a thoroughly first-class tool rather than the absolute last dollar knocked off the first cost of real estate, and therefore, believe the extra cost justified. The engineer can examine the estimate of cost as prepared by the contractor, discuss savings or changes, and can see places where he is willing to cut the design, when he knows that there is not going to be still further cut in the execution of the work. When he knows that the work is going to be executed exactly as designed, he can design with greater precision and economy than when he has to provide extra strength or size of members to allow for a possible lack of quality of materials and workmanship by some unknown person who may have the execution of the work in hand. The work can be carried on also much faster under this method of close co-operation than under the other, where there are likely to be misunderstandings which must be adjusted before the work progresses further. What the contractor has to sell is service, and what the owner wants is results, and where this mutuality exists both parties can accomplish the end for which they are striving.

The conservative contractor figuring on a lump sum contract includes everything shown at a price for which he feels sure he can do the work; then, as every emergency cannot be foreseen, he allows an item for contingencies, and finally adds a percentage for use of capital and profit. Under a fixed profit contract, however, where the contingent risk is assumed by the owner, a lesser profit will be accepted by the same contractor because it is assumed that the total cost may be reduced by co-operation between contractor, engineer and owner in the matter of design and the purchase of materials. Under average conditions a saving as great as 10 per cent may be made. The specialist will obtain lower unit costs on the work than a less experienced man who is liable to submit a lower competitive bid. This is why the same quality of work can usually be done at less cost to the owner under the fixed profit basis and why a very material improvement in quality can be secured with very slight increase in cost.

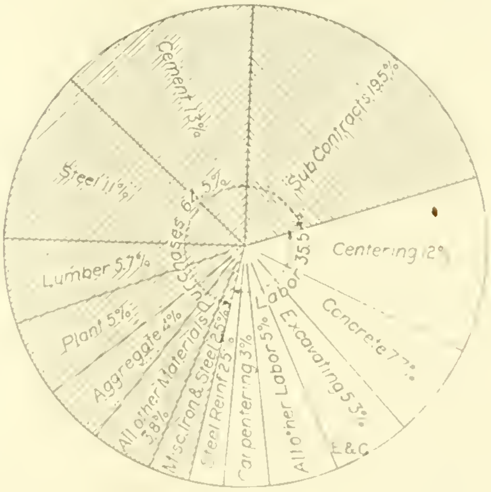


Fig. 1. Diagram Showing Sub-division of Costs of Reinforced Concrete Building Erected by Aberthaw Construction Co. in 1912.

If the owner wants to start as soon as he engages an engineer and before plans are started, he can save about six weeks' time with no serious difference of cost on the cost plus fixed-sum basis.

There are several variations in details with this type of contract. Its merit lies in removing one of the most trying problems of the contractor—his antagonistic relation to engineer and owner.

Because the handling of reinforced concrete is to a large extent a manufacturing proposition, it is obviously more desirable to let such work to specialists than to those experienced only in the erection of ordinary brick or wooden buildings. In the making of concrete a number of crude materials are brought together and treated in such a way as to produce a new one, whereas in the construction of a wooden or a brick building, materials are merely assembled, but are not changed essentially, after they are in place from what they were before.

The engineer and owner do not sufficiently appreciate that the builder or contractor is largely a broker buying materials. With an office building, for instance, the general contractor may not handle more than 10 per cent of the contract price as direct labor. The rest is materials bought and sub-contracts let. With reinforced concrete, the percentage of labor is larger than in any other building operation. It averages about 35 per cent of the cost. The contractor, therefore, realizes the necessity of a first-class purchasing department. Most manufacturers have such a department. Sometimes one and sometimes the other has much better facilities for obtaining the lowest price on materials required in construction. Therefore, the co-operation between the two departments produces the best result to the owner. A diagram of the results obtained by the Aberthaw Construction Co. on a recent building is given in Fig. 1 to illustrate the important divisions of cost.

PRELIMINARY WORK.

While the estimating department is busy scaling plans and computing quantities, the writer or an associate examines the site of the proposed structure, calls upon various parties who can give information useful in the preparation of an estimate; such as municipal authorities, for information as to local regulations, the customs department and immigration bureau, if job is in Canada, dealers in all the principal building materials required, and the contractors for the principal lines of sub-contracts, such as carpentering, plumbing, roofing, painting, sheet metal, bricklaying, etc., also dealers in contractors' machinery of various kinds. He visits the engineers of the local railroad, whose main line adjoins the site of the work, when it is proposed to install a siding, as the time of its completion is important; also the general freight agent in regard to rates on material at this point. General contractors, and all others, who would have any information regarding the labor situation are consulted. From the data thus obtained, and by studying the unit costs of completed jobs which somewhat resemble the one in view, prices are determined for use in an estimate to be submitted to the engineer.

ANALYSIS OF THE DESIGN.

When a contract is awarded, and a set of plans is received from the engineer, a considerable amount of work is done in the main office, beginning long before any work is started in the field and continued during the early stages of construction, in two departments—the designing and the purchasing. The designing department first will be considered.

The function of the designing department is to take the complete set of plans and to make a careful study of these in regard to designing forms, tabulating steel, and illustrating and tabulating other materials, such as doors, windows, architectural iron, etc., where this is not done in the engineer's office. The location of construction and contraction joints is studied, and on rare occasions the structural design is reviewed. In the writer's practice it is common to close a contract and start building before any working drawings are made. Then the work and responsibility thrown on the designing department are very great. Also some of the below-mentioned minor features become major ones and are, therefore, discussed here as though they were.

No attempt will be made to describe ordinary designing for strength, but rather those features which are not usually shown on plans, and others which a contractor desires to change for the sake of economy, simplicity in construction, avoidance of cracks, etc.

Minor Features.—This term is applied to those parts of the designing which are not vital to the construction, and therefore frequently are not shown on plans, being left either to be settled in advance by the builder or during construction by himself and the engineer. Some of these points are learned only by experience in actual construction, and may not have occurred to the engineer until called to his attention.

Chief among these minor features to receive attention are the means to be provided to prevent cracks, which mar the appearance and are often a source of annoyance though seldom a cause of structural weakness. There are, sometimes, cracks caused by unequal settlement of the foundation. Sometimes a part of a building is on piles and part on earth. Where they join, it is hard to foresee and to provide enough bearing so that the whole will settle evenly. It is usually best to provide a broader bearing on the less rigid material than theory dictates, then unsightly cracks may be avoided by very rigid tying of the building together with reinforcement, against pulling apart longitudinally or shearing. This requires both horizontal and diagonal reinforcement with a minimum of at least 0.5 per cent of steel. A somewhat larger amount is preferred.

Another way to overcome the same difficulty is by building a joint vertically entirely through the structure at this point and designing the members of the structure on each side so that, if settlement does occur, they will not crack. This was done with satisfactory results in a recent building where one end rested upon sand, the center on ledge, and the other end on clay.

Large wall surfaces without joints are very likely to crack vertically, due to the shrinkage stresses of the concrete. Experience has shown that thin walls with insufficient reinforcement will crack about every 25 ft. of their length where exposed to the air on both sides. Thick walls will crack about every 50 ft. Where the wall is exposed to the air on one side only, and kept at a fairly uniform temperature, either by water or earth on the other side, there is very much less danger of cracks.

Temperature stresses will not be considered by themselves. The writer has not met a case where the structure could change its temperature so rapidly as to crack from this cause. If properly reinforced against shrinkage cracks, the temperature stresses will be sufficiently provided for.

Walls of buildings 400 ft. in length, which have been built without joints, have not developed cracks. However, the amount of steel used, in the writer's opinion, was not justified on the grounds of economy or in comparison with the result obtained. In such a wall internal stresses are very likely to open diagonal cracks, radiating from corners of windows and doors; therefore, diagonal reinforcement is necessary at such points. This is seldom shown on plans when first received.

On the ground of cheapness of construction, it is almost always better to put in curtain walls after the frame is up; the work then goes ahead much faster. Weather-tight joints can readily be made, and the joints between the frame, i. e., the columns and floor, can be concealed so that no unsightly crack is visible.

Where a floor is built around machinery foundations, frequently a long span and a short span beam come close together. Unless provision is made for a joint between them, there is certain to be an unsightly crack, due to the difference of deflection of the two under the same load

Another point of design which may sometimes be advantageously discussed by the engineer is that of roof construction, whether the structural part shall be flat and covered with cinders, to give a proper pitch for drainage, or whether the ceiling shall be pitched and the fill avoided. In construction the most serious objection to the first method is that so much time elapses between the casting of the roof and the waterproof cover of tar and felt over the cinder fill that a considerable quantity of rain collects on top of the roof slabs. These slabs, while not absolutely watertight, are tight enough to hold a considerable quantity of water. On some jobs holes had to be drilled from the ceiling to drain the roof slab. In one building water dripped from the ceiling after it had been waterproofed for a period of five months, due to the collection of water on the slab in the cinder fill. In another case there was some dripping after two years. This is a matter entirely beyond the control of the contractor, as it is solely a question of weather conditions, and can be avoided only in the engineer's design.

There is a question as to whether stairs shall be cast integrally with the floors or whether rods shall be left projecting from floor to bond the stairs, which are cast later. The latter method is much preferred by the writer.

Another important question which must be taken into consideration in the designing is whether a granolithic floor is put on as an integral part of the construction, or as a second operation, and if a second operation, what its thickness shall be. For economy's sake, both in materials and in labor, it is cheaper to put on the finish with the construction. Under these conditions, it is frequently impossible to keep off of the floor long enough for it to harden sufficiently to prevent its being somewhat marred. If this is a serious objection, the finish must be put on as a second operation. As this does not bond sufficiently with the floor to be considered a unit, the construction should be thick enough to carry the load without any assistance from the finish. If the finish is 1 in. thick, it is usually safe for the contractor to guarantee that it will be loose in spots, whereas if it is 2 ins. thick it is safe to guarantee it as solid and satisfactory. This adds to the dead weight and to the expense. It is necessary to settle these points before construction begins in order to get the minimum cost and the maximum efficiency, as the amount of forms bought and the conduct of the work are influenced by them.

Frequently, the location of construction joints between different days' work is an item of importance, but these are seldom indicated on plans, and therefore must be discussed and settled. Some of the points which influence their location are, the capacity of the mixing plant, the supply of materials, and the weather. Sometimes the latter causes several days' delay where continuous work is desired. There is a question as to whether the construction joint shall be in the middle of the span of a floor, or be made in line with the columns. The treatment in the two cases is somewhat different. If in line with the columns, these must be thoroughly reinforced above and below the floor, using steel plates, otherwise they are certain to be split when the floor shrinks at the construction joint. If the building is to have contraction joints, the construction joints should be made so as to coincide with them.

The cleaning down and finishing of the exterior of a building, according to the type of finish specified, whether it is left as it comes from the molds with just the bad places rubbed or whether it is to be tooled or plastered, will in some degree affect the design of forms, and therefore must be considered before work is begun.

The designing department of the writer's company has several times found defects in plans, such as hanging of large three-ply fire doors in an 8-in. brick walls, where the weight of the doors is greater than that of the brick work. The writer recommends that the iron door jambs be carried straight through from floor to ceiling and anchored to the ceiling to keep the doors in place.

It is somewhat common to put wall beams above the floor on account of letting in light at the ceiling line. Then the question arises whether the beams shall be cast with the floor or shall be cast separately. It is generally cheaper and of equally good construction to cast the wall beam as a separate operation, allowing the ends to rest in recesses in the columns, and to suspend the slab below by means of stirrups. Provision can be made for continuity by casting some holes in the columns above the floor at the proper points, through which the reinforcement for negative bending moments will pass, allowing the ends to extend well into the adjacent panels. When properly grouted into place and when the panel is cast this will be as strong as if cast as a monolith with the floor.

Because concrete is a plastic material cast in place, and because the molds can be made of any design, engineers seem to feel that there is no necessity for holding to any standard sizes for the members of a concrete structure, although they do when designing in steel. An engineer is justified in giving considerable thought to the selection of sizes, as the particular dimensions which best satisfy his designs are not always the most economical. In one building of large size, in a single floor there were in the original design 455 different sizes and styles of beams. The designing department of the writer's company found it possible, without impairing the result to be accomplished, to reduce these to less than a tenth of the original number, and by co-operation with the engineers finally succeeded in reducing the number to 52. *If the engineers realize that in the building of wooden forms for maximum economy these are made, from plans drawn to scale, at a bench on the ground and assembled in place without the use of any tool but a hammer, it will be seen that almost as great precision must be used in their construction as with structural steel.*

The question of change of size of columns in every story, or once in two or three stories, is one of cost. The expense of changing the forms is usually greater than the cost of the concrete saved on small columns where the reduction is less than 3 ins. and on large columns where the reduction is less than 2 ins. Therefore, it is not often economical to change the size every story. Often when the column is reduced it is desirable to use ahead the size of lowest story columns in order to avoid expense of splicing out beam and girder at sides and bottoms. Sometimes when the upper floor has a lighter load than the lower, it would be possible to reduce the size of beams. Economy dictates that the depths may be reduced but not their width, because this would require splicing out all the floor panels and joists, which is more expensive than the saving in concrete.

Lastly, the question of whether the floor panels may be centered with wood or corrugated iron may be discussed, and must be settled very early. In the writer's opinion, the appearance of corrugated iron ceilings is better than that of lumber, and it is just as easy to attach inserts to the mental forms. They are somewhat more economical.

Uncommon Features.—In a certain mill, the basement floor was partly of beam construction with concrete slab and partly of concrete beams without slab, the floor being made of wood, as the owner desired an opportunity for frequent changes of the arrangement of his mechanical plant. This caused special form-work to be used, which could not be used again, and also made it difficult to support the floor above.

The frequency of special features in a lower floor, and the certainty that the basement story height is less than that above, adds to the cost of form work. This is at a time in the process of construction when new lumber must be cut up although it could be advantageously used in long lengths on the upper stories. If it cannot be used again, it has to be remade, and the remaking of second-hand lumber into new shapes is more expensive than when it is new.

Where extremely heavy loads are to be carried on columns, sometimes so much steel reinforcement is shown that it is impossible to use stone concrete, as the bars are so close together that the column really has to be filled with mortar. This is doubtless an oversight on the part of the drafting room of the designer, and makes a problem for the contractor to adjust before work can progress. Similarly, heavy girders have had so much reinforcement that there was not enough concrete in the width of the girders shown to imbed the reinforcement, as shown on plans.

An unusual problem put up to the writer's company to solve in execution was a very high tower which had two floors near its top, one 160 ft. and the other 180 ft. from the ground, each of which carried a load of 400 tons on an area of 28 by 30 ft. This floor was supported on a central girder, running the short way, with beams running from it to the opposite walls. The requirement of engineer was that the walls and floors be cast as a monolith and be watertight as a tank, as moisture penetrating from driving storms would be a serious handicap to the operation of the plant. This unusual feature, high in the air, required absolute prevention of all cracks from any cause, impermeable concrete and as near continuous work as possible. Under ordinary conditions there would be frequent joints between days' work of reasonable size which might allow the penetration of the weather. This problem was solved by a little lower working stresses in materials than is common, by a rich mixture carefully proportioned for a maximum density by designing forms for casting a large amount of concrete at one time, and by continuous work with different shifts of men, combined with very close supervision by more than the usual number of bosses, besides the inspector. The result proved satisfactory in every way.

Method of Drawing Details.—The preparation of drawings for details of steel and form work is very similar. They have been standardized as far as possible to simplify the work in the drafting room, and also so that the men handling the plans outside can easier understand them.

Columns are shown full height from footing to roof, showing the outline which is to be built in forms, the joints with curtain walls and floors, as they may occur, and the steel reinforcement shown by as simple a method as can be devised. Each beam is shown by itself inside elevation, with sections and notes as to the number and location.

It is seldom necessary to make an assembly drawing, as the engineer's plans are sufficient for this purpose, but while the details of some engineers are sufficient, they are as a rule reduced to a system which experience has shown to be the most useful to the men on the work.

Water Works

Design of the Proposed Aqueduct for the Additional Water Supply of Winnipeg, Manitoba.

An article published in the current news section of Engineering and Contracting of Nov. 5, 1913, presented some of the salient features of the proposed works for the additional water supply of the city of Winnipeg. That article gave a schedule of the required works and the corresponding estimate of cost. The conditions under which the work will be prosecuted were outlined and other information of interest to contractors was given. The present article relates almost wholy to the engineering features of the proposed aqueduct, the information presented being abstracted and quoted from the report of the Board of Consulting Engineers, Messrs. Rudolph Hering, Frederick P. Stearns and James H. Fuertes.

The new supply will be drawn from Shoal Lake, one of the lakes in the same group as the Lake of the Woods, and will be conducted to the city through an aqueduct of total length of 95.35 miles. The inlet to the aqueduct will be near the west end of Indian Bay, an arm of Shoal Lake. Leaving the lake the water will pass through a concrete aqueduct 84.75 miles in length, laid with a continuous down grade to a point about a mile east of Transcona, a suburb of Winnipeg, thence through a 5-ft. steel pipe line to the Red River, thence under the river through a 5-ft. cast iron pipe in tunnel, and thence through a 4 ft. cast iron pipe laid in the city streets, to the reservoirs at McPhillips St. The concrete portion of the aqueduct will be given a capacity of 85,000,000 imp. gals. per day and the pipe line portion, 25,000,000 gals. per day. The construction of a 250,000,000-gal. reservoir at a point on the aqueduct about a mile east of Transcona is recommended, though not for immediate construction, and it is further recommended that a main pumping station be then established to force water to the city through the 5-ft. steel pipe. This reservoir and pumping station should be completed and ready for use before the demands for water have reached the capacity of the pipe line portion of the aqueduct.

QUANTITY OF WATER REQUIRED.

The population of the Greater Winnipeg Water District is about 225,000, about 200,000 being in the city of Winnipeg. This population is twice that of seven years ago and five times that of 15 years ago. The average per capita consumption of water increased from 32 imp. gals. in 1902 to 47 gals in 1912. The increase per capita is, of course, likely to continue with the increase in facilities for using water.

It is believed that 85 gals. per inhabitant is a proper quantity to adopt for estimating the future needs of the district as an average for the year, but in some months the rate will average as high as 100 gals. per inhabitant. Therefore an aqueduct to provide for a considerable time in the future should be designed with a capacity of 100 gals. per inhabitant, and even with this provision a large reservoir should be built near the city, in the future, to provide for a still higher rate of water consumption during short periods.

The consumption of water at the present time is somewhat restricted by the limited capacity of the sources of supply and will undoubtedly increase to a considerable extent when a new supply with a better water is furnished. For the next 10 years, however, the consumption is likely to be less than 85 gals. per inhabitant.

The best conduit for conveying water for the greater part of the way from Shoal Lake to Winnipeg is a concrete aqueduct laid with a continuous down grade, and as such a structure can be built of large capacity for a moderate additional cost, the engineers have designed this structure to carry 85,000,000 gals. in 24 hours, sufficient, on the basis assumed, for a supply throughout the year for 850,000 inhabitants. While this is a large provision for future years, it should be borne in mind that the additional cost of making such a structure of liberal size is comparatively small while the cost of duplicating such an aqueduct would be large.

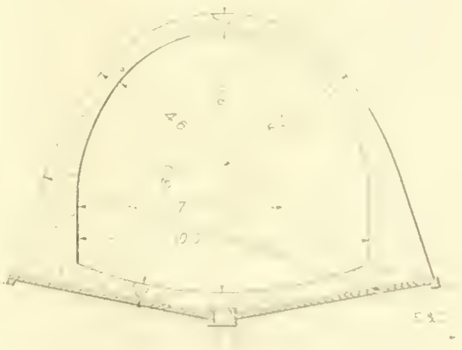


Fig. 1. Cross Section of 9x10 ft. Portion of Proposed Aqueduct for Additional Water Supply of Winnipeg, Manitoba.

Area of section, 73.6 sq. ft.
Perimeter, 31.6 ft.
Hydraulic radius, 2.33 ft.
Coefficient adopted, 132.8.
Concrete per lineal foot, 1,156 cu. yds.

The capacity of 85,000,000 gals. daily was adopted after giving due consideration to the many demands for money for public works in a growing city like Winnipeg, otherwise a larger aqueduct, say one having a capacity of 100,000,000 gals. in 24 hours would have been recommended.

From the point near Transcona where the concrete aqueduct ends, to the existing McPhillips St. reservoirs, a pipe is recommended, and a single pipe line of the sizes which now seem desirable will convey by gravity 25,000,000 gals. of water in 24 hours, or nearly three times the quantity of water now used. When necessary, the flow through this pipe can be increased by pumping, or a second pipe can be laid.

Assuming a continuous growth for the Greater Winnipeg Water District, based upon the past growth of the district and the rate at which other somewhat similarly situated cities have grown, the 85,000,000-gal. aqueduct would have sufficient capacity for supplying the district until 1944. Assuming further the introduction of water from Shoal Lake at the earliest practicable date, the aqueduct would supply the requirements of the district for about 25 years after its completion, and the 25,000,000-gal. pipe line would supply by gravity the quantity of water required for two or three years; but, as already stated, the capacity of the pipe line can be increased by pumping.

THE AQUEDUCT.

Intake.—The intake of the aqueduct is to be located at a corner of the bay where conditions are favorable in that the shore for a long distance is rocky, and the depth of water 50 ft. from the shore is 10 ft. and 170 ft. from shore is 15 ft.

It is proposed to build the gate and screen chamber on the shore and to protect the intake from material which may drift along the shore during winds, by building piers extending at least 150 ft. into the lake on each side of the intake, or by laying a large submerged conduit to bring the water from a point at least 150 ft. from the shore. The conduit is preferable, if it can be built for nearly the same cost as the piers. The bottom of the aqueduct at the bay is 11 ft. below high water.

The gate and screen chamber should be designed with liberal areas for sluice gate openings and for screens, so as to cause as little fall of the water from the bay to the aqueduct as practicable. There should be at least two sluice gates not less than 5 ft. wide and 6 ft high, and the screens should have a total length of not less than 50 ft. and a height extending from the bottom of the aqueduct to the surface of the water.

Aqueduct Designs.—In order to take water from Indian Bay by gravity, the aqueduct for the first nine miles has to pass through land that is higher than Indian Bay, thus necessitating a deep summit cut. After passing this nine miles, a line can be selected which will be down grade all the way to Transcona, but the gradient obtained for an aqueduct varies from place to place, and consequently the size of the aqueduct required to convey a given quantity of water also varies.

The engineers, therefore, prepared a series of designs of horseshoe shaped aqueducts ranging from the large one, 9 ft. high and 10 ft. wide, used at the summit cut where there is a low gradient, here shown in Fig. 1, to one 5 ft. high and 5 ft. wide, used where the gradient is highest. Aqueducts of this kind are expected to flow partly full as a rule and never to have any considerable pressure of water against the top of the arch.

For the first 26,000 ft. of the concrete aqueduct, near Transcona, it is desirable to use a section which can stand internal pressure, as it will be necessary at some time in the future to build a large reservoir at the end of the concrete aqueduct, in which the high water level may be 18 or 20 ft. above the surface of the ground and the aqueduct will be, at least in part, below the surface of the ground. This portion of the aqueduct has been designed of circular form, 8 ft. in diameter and provided with steel reinforcement. This pressure aqueduct will be equally valuable before the reservoir is built, as it will provide additional head at the end of the steel pipe to force the water through the pipe to the McPhillips Street reservoirs.

At the crossing of the Brokenhead River and the lowland each side of it, for a total distance of 13,000 ft., it has also been found desirable to adopt the circular form of aqueduct reinforced with steel so that this portion will withstand the pressure resulting from building it 10 ft. or more below the regular grade line.

As the aqueduct, with the exception of the reinforced portions above mentioned and some other short lengths at river crossings, is not expected to stand any considerable upward pressure on the arch, measures must be taken to prevent the opening of the sluice gates at Indian Bay to a height which will permit a quantity of water to enter the aqueduct much, if any, in excess of the rate of 25,000,000 gals. daily, which is the limit of the capacity of the pipe line leading from Transcona to the city reservoirs, and as a further safeguard waste weirs should be provided at several places where the aqueduct crosses streams to permit the water to overflow into such streams if it should rise above the safe elevation.

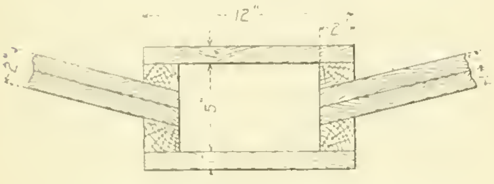


Fig. 2. Part Section of Box Drain and Timber Foundation—Winnipeg Aqueduct.

In order that the aqueduct may be both watertight, strong and permanent, it is recommended that it be built of first-class concrete mixed in the proportion of one part of Portland cement to two parts of sand and four parts of screened gravel. To prevent transverse cracks in the aqueduct, it should be built in sections not more than 20 ft. in length, and where one section joins another a steel bar about 3 ins. by ¼-in. should be inserted to act as a water stop and prevent leakage at these joints, or some other suitable water stop should be provided.

The aqueduct cross-section shown in Fig. 1 has a central wooden box drain and a platform composed of two thicknesses of 1-in. boards, the upper one being fluted on the under side to furnish channels in which the water may flow from the side of the trench to the drain. The only purpose of this platform is to furnish a dry bottom upon which the concrete bottom of the aqueduct may be built and kept dry until it has had time to set. The dimensions of the box drain are shown in Fig. 2. All sections of the aqueduct may be provided with this platform.

Although a double platform is shown it is better where there is but little water to be taken care of to use only one board with grooves in each edge of each board through which the water can pass to the central drain. Both types of plank foundation are shown in Fig. 3. The boards should be bedded in sand. If there are places along the line where a platform will not be kept continuously wet after the aqueduct is completed, the platform should be omitted because of the liability to decay under such circumstances.

[illegible] and Embankment.—Where pipes are to be laid in the streets, they are to be placed at a depth which will permit a 4-ft. covering of earth. East of Transcona, where the aqueduct will not follow the streets, but be carried through the open country, the much cheaper and more expeditious plan will be adopted of building the aqueduct in a shallow trench—from 3 to 4 ft. deep, where practicable—and covering it to a depth of 4 ft. with an embankment of earth.

It is proposed to make this embankment with a top width of not less than 8 ft. and nowhere less than the inside width of the aqueduct and to make the slopes of the embankment 2 horizontal to 1 vertical.

Aqueduct Statistics.—Table I gives various statistics relating to the different portions of the aqueduct, including the lengths, gradients, elevations and dimensions of the different portions.

TABLE I.—LENGTHS, SLOPES, ELEVATIONS AND DIMENSIONS OF THE VARIOUS SECTIONS OF THE AQUEDUCT FROM THE McPHILLIPS STREET RESERVOIRS IN WINNIPEG TO SHOAL LAKE.

Total Distance from McPhillips St. Reservoirs. Feet.	Miles.	Length of section, ft.	Dimen'ns of Aqueduct. Height, ft.	Width, ft.	Slope in ft. per 1,000 ft.	Elevation of hydraulic gradient at end of section, ft.	Remarks.
0	0	......	....	....	....	41.31	High water in McPhillips St. Reservoirs.
12,000	2.27	12,000	4 ft. diam.	C. I. pipe	0.81	51.03	These slopes and elevations are for a discharge through the pipes of 25,000,000 gals. daily.
56,100	10.62	44,100	5 ft. diam.	Steel pipe	0.37	67.35	
82,100	15.55	26,000	8 ft. diam.	Rein. conc.	0.025	68.00	
56,100	10.62	......	....	....	....	61.00	Proposed high water in reservoir east of Transcona.
82,100	15.55	......	8.0	8.0	0.29	68.54	This slope based on 85,000,000 gals. daily.
						Elevation of inside of aqueduct bottom at end of section.	
82,100	15.55	......	...	...	....	83.0	Beginning of grade conduit.
113,100	21.42	31,000	7.5	8.5	0.32	72.92	....
128,100	24.26	15,000	6.4	7.4	0.70	83.42	....
141,600	26.82	13,500	5.7	6.4	1.23	100.02	....
156,100	29.57	14,500	5.1	5.1	2.83	141.05	....
165,100	31.28	9,000	5.8	6.7	1.00	150.05	....
229,700	43.54	*64,600	6.6	7.6	0.54	184.93	....
246,500	46.69	16,800	5.3	5.7	1.90	216.90	....
262,500	49.71	16,000	6.7	7.7	0.50	224.90	....
306,500	58.65	44,000	7.4	8.4	0.30	238.10	....
392,500	74.34	86,000	6.3	7.3	0.69	297.44	....
445,400	84.35	52,900	7.3	8.3	0.32	314.37	....
455,400	86.25	10,000	8.0	9.0	0.24	316.77	....
470,400	89.09	15,000	9.0	10.0	0.24	320.37	....
503,359	95.34	32,959	9.0	10.0	0.11	324.00	....
503,400	95.35	41	...	....	0.00	324.00	Gate and screen chamber.

*13,000 ft. of this length at Brokenhead River Crossing is 7.2 ft. in diameter and reinforced.

For a distance of 33,000 ft. from Indian Bay, the gradient of the aqueduct has been made only .11 ft. in 1,000 ft. This low gradient was adopted to diminish the depth of the summit cut. The aqueduct through this section is 9 ft. high and 10 ft. wide.

For the next 25,000 ft. the fall per 1,000 is .25 ft. The large sized aqueduct, 9 ft. high and 10 ft. wide, is continued for 15,000 ft. of this distance and a smaller aqueduct, 8 ft. high and 9 ft. wide, for the remainder of the distance. This design is the result of somewhat complicated hydraulic computations which show that with these sections of the aqueduct as planned 85,000,000 gals. daily will flow through the aqueduct when the water in the bay is 3.75 ft. below high water level, provided the water has such a free entrance through the intake and gate and screen chamber that it does not fall more than .25 ft. in passing through them.

Beyond the three portions of aqueduct above described, the gradient of the aqueduct has been made to follow approximately parallel with the surface of the ground all the way to its end near Transcona, and a size has been adopted for each gradient, with a few exceptions which would give the aqueduct the standard capacity of 85,000,000 gals. per day.

The quantity of water which will flow through the reinforced section of aqueduct, used for the last 26,000 ft., will not depend upon the gradient of the aqueduct itself, but upon the hydraulic gradient available when a reservoir is built east of Transcona, and its size has been computed accordingly.

Somewhat similarly, the next two portions of the aqueduct east of the reinforced portion, covering a total length of 46,000 ft., have been made a little larger than the size due to the gradient of these portions, on account of the possible back water effect from the reservoir.

The flow through the pipe line is, of course, dependent upon the hydraulic gradient and not upon the gradient of the pipes. For a flow of 25,000,000 gals. daily, this amounts to .37 ft. per 1,000 in the 5-ft. pipe and .81 ft. per 1,000 in the 4-ft. pipe, making a total fall through these pipes to the McPhillips Street reservoirs of 26 ft.

Red River Crossing.—The designs provide for crossing Red River with a pipe placed in a tunnel, the tunnel extending about 900 ft. horizontally across the river from a shaft in Victoria Park to a corresponding shaft on the opposite side of the river. The shafts will each be about 75 ft. deep. After considering different plans, one plan was selected which provides for the use of cast-iron pipes 5 ft. in diameter, with metal 1.9 ins. thick, provided with a special bell about 15 ins. long, so designed as to permit the joint between pipes to be caulked from the inside with lead wool. The tunnel is to be excavated only large enough to permit the pipe to be put in place. This requires that its minimum diameter shall not be less than 6 ft., 3 ins. and the average diameter, owing to the irregularities in blasting, will be considerably more. All spaces between the pipe and the sides of the tunnel are to be filled with concrete, as far as it is practicable to place concrete, and the remaining space is to be filled by pumping in cement grout, the aim in using such thick pipe and in filling all spaces with concrete or cement grout being to make absolutely secure this portion of the pipe line which will be inaccessible unless the

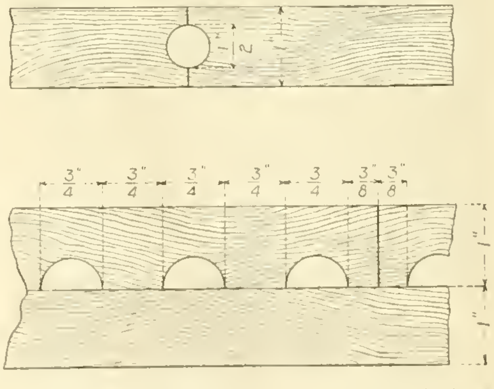


Fig. 3. Detail Section of Plank Foundations Where One and Two Thicknesses Are Required—Winnipeg Aqueduct.

water is shut off and the pipe is emptied by pumping.

Seine River Crossing.—The pipe where it crosses the Seine River is located in a proposed street, and the designs provide for building a 30-ft. arched concrete culvert, filling over it with earth and laying the steel pipe in the earth in the same manner as in the other places.

Other Special Structures Along Aqueduct.—In addition to the intake works at Indian Bay, special construction and structures will be required at various places. At the crossing of the Falcon River a pile foundation will be required for about 600 ft., and for about 150 ft. of this length it will be necessary to depress the aqueduct so that its top will be low enough to permit the passage of the water of the river and of boats of light draft. This depressed portion should be reinforced with steel, and the concrete should be made much thicker than under ordinary circumstances. In a similar way there should be a depressed and strengthened section of aqueduct at the crossing of the Whitemouth and Brokenhead Rivers, also at the Birch River, unless it is found to be feasible to carry the aqueduct over this river. For a part of the distance at the crossing of the Brokenhead River, where the muskeg is too deep to permit of the construction of the aqueduct upon a solid natural bottom, a pile and timber foundation will be required.

Blow-offs should be provided for discharging water from the aqueduct into Boggy, Birch, Whitemouth, Brokenhead and Red Rivers and into other places where an opportunity offers, such blow-offs being convenient for use during the construction of the aqueduct and while it is being cleaned or repaired and for use in emergencies when the aqueduct is in service.

Waste weirs should be provided at the Birch, Whitemouth and Brokenhead Rivers and provision should be made at these waste weir structures for placing stop planks across the aqueduct to stop or throttle the flow through it. By making suitable arrangements, it will be practicable to force over the waste weirs any excess of water above the capacity of the pipe line leading from Transcona to the city which might accidentally be permitted to enter the aqueduct at Indian Bay.

Culverts will be required where all small streams cross the aqueduct line and at other places where there are no definite streams, in order to permit water to find its way from one side of the aqueduct to the other. All culverts should be so designed that they will not become plugged by ice during the winter.

PERMANENCE OF CONCRETE MASONRY.

Concrete masonry is under most circumstances one of the most durable and permanent of materials, but in certain soils and in the presence of sulphates and sulphuretted hydrogen it sometimes softens and disintegrates. Even under these adverse circumstances it has been found that a dense, smooth and impervious concrete is very resistent to deterioration.

In order to ascertain whether or not any deterioration of the concrete of the proposed aqueducts is likely to take place, the engineers had certain chemical analyses made of the muskeg waters, the materials forming the muskegs and the underlying soils, both in the Laurentian formation, which underlies the greater part of the length of the aqueduct, and limestone formation which occurs towards its westerly end. The results of these analyses show that the sulphates and sulphuretted hydrogen were not present in the Laurentian formation, and were there only in small quantities, in the limestone.

As already stated the policy adopted provides for building the aqueduct, where possible, in a shallow trench from 3 to 4 ft. deep. This method of construction will permit of the drainage of the muskegs and other ground in the limestone formation, nearly as low as to the bottom of the aqueduct, so that very little of the ground water, which may contain sulphates and sulphuretted hydrogen, will come in contact with the concrete, and the engineers recommend that especial attention be given to providing good drainage along the aqueduct through the limestone formation.

Although the engineers believe that with concrete mixed in the usual proportions there would be very little danger of disintegration they have thought it best to recommend that the concrete be made of such proportions of cement and other materials that it will be especially dense and strong, and therefore much less likely to be injuriously affected.

They are convinced that the concrete aqueduct, if built as recommended, will be a permanent structure.

A Complete Accounting System for Both Large and Small Water Works Properties.

Contributed by Le Roy W. Allison, Newark, New Jersey.

A general, comprehensive accounting system for any specific utility must necessarily offer great flexibility to afford adequate service for both large and small operating companies. Yet as a whole the accounting schedule would comprise the same general standard classifications, with elimination of such intermediate sub-classifications as would be non-operative and accordingly unessential with the smaller organization.

The components of a thorough accounting system include a schedule of accounts of Assets, Liabilities, Fixed Capital, Income, Operating Revenue and Operating Expense, and, for facilitation, would be divided into two great classifications under the headings of Balances or Indicant Accounts and Income Accounts. Of these, the first would comprise Asset, Liability and Fixed Capital Accounts; and the second, Income, Operating Revenue and Operating Expense Accounts.

A comparison of the balances of the Indicant Accounts would show the existing condition of a company at any specific time, and constitutes, with the subsidiary schedules, all that is necessary prior to that time when the company may be termed an active organization.

Effective with operation, the second group, or Income Accounts, becomes necessary, thereby recording in convenient form the accounting history of its various activities. This schedule closed upon a basis of Corporate Surplus or Deficit, yearly or otherwise, connects both groups of accounts.

The accompanying system of accounts for water works properties, arranged in concise and convenient form with explanation of classifications where essential, offers a uniform and complete schedule for efficient application. Based upon systems approved and prescribed by public utility commissions, and thus standardized in accord with each particular feature of operation, it is adaptable to both large and small properties, with any required elimination in the latter case as above mentioned.

TABLE I.—ASSET ACCOUNTS.

(1) Fixed Capital Installed.
(2) Fixed Capital Installed in Other Departments.
(3) Construction Work in Progress.
(4) Materials and Supplies.
(5) Current Assets
  (I) Cash.
  (II) Bills Receivable.
  (III) Acc'ts Receivable
  (IV) Interest and Dividends Receivabe
  (V) Other Current Assets.
(6) Coupon Special Deposits.
(7) Other Special Deposits.
(8) Investments.
(9) Re-acquired Securities.
(10) Sinking Funds.
(11) Other Special Funds.
(12) Prepayments.
(13) Unamortized Debt Discount and Expense.
(14) Property Abandoned.
(15) Other Suspense.
(16) Corporate Deficit

EXPLANATION OF ASSET SCHEDULES SHOWN IN TABLE I.

(2) Fixed Capital in Other Departments: Tangible and intangible, devoted to other than water operations.

(3) Construction Work in Progress: Expenditures upon plant and equipment in process of construction, but not ready for service; including consistent proportion of expense for supervision, engineering, tools and supplies; upon completion, cost to be credited to this account and charged to the proper Fixed Capital Account.

(4) Materials and Supplies: All materials and supplies acquired, salvage value of discarded equipment, and of equipment, materials and supplies returned to store, regardless of whether same are to be consumed in construction or in operation, or later to be sold.

(5) Current Assets, (V) Other Current Assets: To provide for all current assets not includible in other classifications under this schedule.

(7) Other Special Deposits: All funds and bank credits deposited in the hands of agents for other special purposes, and not elsewhere provided for.

(11) Other Special Funds: All such belonging to or held in trust for specific purposes, and not otherwise provided for.

(12) Prepayments: Advance payments for taxes, insurance, rents, etc., prior to the period to which such pertain.

(15) Other Suspense: All expenses of preliminary surveys, plans, investigations, etc., made for determining the feasibility of projects under contemplation. Through the carrying to completion of any such later on, the amounts so expended to be credited to this account, and charged to the proper Fixed Capital Account; if later abandoned, the amounts to be charged to the appropriate Income or Operating Expense Account.

(16) Corporate Deficit: To show the debit balance, if any, in the Corporate Surplus or Deficit Account.

TABLE II.—LIABILITY ACCOUNTS.

(1) Funded Debt Account.
(2) Unfunded debt accounts...
  (I) Taxes Accrued.
  (II) Judgments Unpaid.
  (III) Receiver's Certificates.
  (IV) Interest Accrued.
  (V) Other Accrued Liabilities.
  (VI) Dividends Declared.
  (VII) Bills Payable.
  (VIII) Advances from Other Companies.
  (IX) Consumers' Deposits
  (X) Other Accounts Payable.
  (XI) Unearned Revenue.
  (XII) Other Unfunded Debt
(3) Premiums on Stocks.
(4) Other Permanent Reserves.
(5) Contractual Reserves.
(6) Accrued Amortization of Capital.
(7) Unamortized Premium on Debt.
(8) Other Required Reserves.
(9) Casualty and Insurance Reserve.
(10) Other Optional Reserves.
(11) Capital Stock Accounts.
(12) Corporate Surplus.

EXPLANATION OF LIABILITY SCHEDULES SHOWN IN TABLE II.

(1) Funded Debt Account: To be divided into classes, such being determined by the four characteristics—(a) Mortgage or other lien or security therefor; (b) Rate of interest; (c) Interest dates; (d) Date of maturity. A separate sub-account to be carried for each such class of funded debt.

(2) Unfunded Debt Accounts, (V) Other Accrued Liabilities: Those accruing from day to day but not coming due until a particular date, such as interest, rent, taxes, etc. (VIII) Advances from Other Companies: All amounts of such advances received to pay for construction or additions and betterments to plant and equipment, or for any other purpose. (X) Other Accounts Payable: All amounts owing to miscellaneous creditors on open account not elsewhere provided for. (XI) Unearned Revenue: All payments received for water to be supplied in future months; as the advance term expires, the proper Revenue Account should be credited, and this account debited with the amount applicable to the time. (XII) Other Unfunded Debt: All unfunded debt upon which the company is liable, and not elsewhere provided for.

(4) Other Permanent Reserves: All reserves created to remain intact until the dis-

solution of the company, not elsewhere classified.

(5) Contractual Reserves: All Reserves necessitated by contracts of the company, such as to cover sinking funds provided for in mortgages, etc.

(6) Accrued Amortization of Capital: Such amounts as are charged from time to time to Operating Expenses or other accounts to cover depreciation of plant and equipment, and other amortization of fixed capital.

(7) Unamortized Premium on Debt: To include the excess of cash value received when funded debt securities or other evidences of indebtedness are disposed of for a consideration greater than the sum of the par value.

(8) Other Required Reserves: All required reserves not elsewhere provided for.

(10) Other Optional Reserves: All such reserves not otherwise classified, a sub-account being carried for each particular reserve.

(12) Corporate Surplus: To show the credit balance, if any, in the Corporate Surplus or Deficit Account.

EXPLANATION OF FIXED CAPITAL SCHEDULES GIVEN IN TABLE III.

(1) Intangible Fixed Capital (I) Organization: All office and other expenses, legal, promotion and otherwise incident to the organization of the company, and placing it in readiness to operate. (IV) Other Intangible Water Capital: All other property coming within such definition devoted to water operations.

Under the succeeding classifications, "Water Collecting System," is embodied all structures and equipment used in collecting water and conveying it from such place of collection to the purification or pumping plant; or in case of a gravity system, to a point from which it is distributed to consumers.

(2) Water Collecting System, (IV) Lake and River Cribs: All land and lake or river bed, and all structures thereon used for collecting water for supply purposes. (VIII) Gravity Intakes and Suction Mains: All land and rights-of-way and structures used for bringing water by gravity or suction from the source of supply to the purification or pumping plant.

Under the next classification, "Purification System," is incorporated all land, structures and equipment engaged in the purification of water, in conveying water from one section of the purification system to another, and including steam or other power plant and pumping equipment, when such apparatus is used in connection with the delivery of wash water for filter or sand washing. It does not, however, include any such steam or other power plant and pumping apparatus which is employed for pumping the water supply from one part of the purification works to another.

(3) Purification System, (II) Coagulating Basins: All land and structures used for the coagulation of water, whether same is preliminary to supply or in conjunction with further purification by some additional process; or whether employed as a step in purification following sedimentation; also, such equipment as coagulant mixing and storage tanks, orifice boxes and feeder apparatus, etc. (IV) Filters: All land and structures used for filtration purposes, whether such filters be "roughing filters," "slow sand filters," or "mechanical filters." and appliances forming a part of such filters, cleaning and sand-washing equipment, etc. (V) Ozone Sterilizing and Aeration Plant: All land and equipment used for the process of water purification by such system. (VII) Clear Water Basins: All land and structures used for the storing of water either in process of purification or as an intermediate step in the purification process, where some particular system or method of purification is employed.

(4) Pumping System, (VI) Miscellaneous Pumping Equipment. All equipment for such purpose not properly classed elsewhere.

(5) Distribution System, (I) Transmission Mains and Accessories: All land and rights-of-way, and all main pipe lines, special castings, valves, valve vaults and boxes, etc., belonging to the transmission system, and accessories to the transmission mains.

The transmission lines, noted above, are

TABLE III.—FIXED CAPITAL ACCOUNTS.

(1) Intangible Fixed Capital.	(I) Organization. (II) Franchises. (III) Patent Rights. (IV) Other Intangible Water Capital.
(2) Water Collecting System	(I) Water Diversion Rights. (II) Reservations. (III) Impounding Dams and Reservoirs. (IV) Lake and River Cribs. (V) Springs and Wells. (VI) Infiltration Galleries and Tunnels. (VII) Collecting Conduits and Reservoirs. (VIII) Gravity Intakes and Suction Mains. (IX) Aqueduct and Supply Mains.
(3) Purification System	(I) Settling Basins. (II) Coagulating Basins. (III) Softening and Iron Removal Plant. (IV) Filters. (V) Ozone Sterilizating and Aeration Plant. (VI) Chemical Treatment Plant. (VII) Clear Water Basins.
(4) Pumping System	(I) Pumping Stations. (II) Steam Power Pumping Equipment. (III) Hydraulic Power Pumping Equipment. (IV) Electric Power Pumping Equipment. (V) Gas Power Pumping Equipment. (VI) Miscellaneous Pumping Equipment.
(5) Distribution System	(I) Transmission Mains and Accessories. (II) Storage Reservoirs, Tanks and Standpipes. (III) Distribution Mains and Accessories. (IV) Service Pipes and Stops. (V) Meters, Meter Boxes and Vaults. (VI) Fire Hydrants and Fire Cisterns. (VII) Fountains, Troughs and Other Equipment
(6) Miscellaneous Properties	(I) General Structures. (II) Gen'l Equipment (a) Office Equipment. (b) Shop Equipment. (c) Store Equipment. (d) Stable Equipment. (III) Other Intangible Water Capital.
(7) Miscellaneous Construction Expenditures	(I) Engineering and Superintendence. (II) Law Expenditures During Construction. (III) Injuries During Construction. (IV) Taxes During Construction. (V) Miscellaneous Construction Expenses. (VI) Interest During Construction.

TABLE IV.—INCOME ACCOUNTS.

(1) Non-Operating Revenues	(I) Rents Accrued from Lease of Water Plant.	
	(II) Miscellaneous Rent Revenues	(a) Leasehold Revenues. (b) Other Rent Revenues.
	(III) Interest Revenues	(a) Interest from Investments. (b) Int'st from Other Sources.
	(IV) Divided Revenues.	
	(V) Miscellaneous Non-Operating Revenues.	
	(VI) Non-Operating Revenue Deductions	(a) Rent Expense. (b) Interest Expense. (c) Dividend Expense. (d) Miscellaneous Non-Operating Expense. (e) Non-Operating Taxes. (f) Uncollectable Non-Operating Revenues.
(2) Income Deductions	(I) Interest Accrued on Funded Debt.	
	(II) Other Interest Deductions.	
	(III) Rent for Lease of Other Water Plant and Equipment.	
	(IV) Miscellaneous Rent Deductions	(a) Joint Facility Rents. (b) Miscellaneous Rent Deductions.
	(V) Miscellaneous Deductions from Income	(a) Sinking Fund Accruals. (b) Guaranties of Periodic Payments. (c) Loss on Operations of Others. (d) Other Contractual Deductions from Income.
	(VI) Amortization Deductions	(a) Amortization of Landed Capital. (b) Amortization of Debt Discount and Expense. (c) Amortization of Premium on Debt.
(3) Appropriation Accounts.	(I) Bad Debts Collected.	
	(II) Other Additions to Surplus.	
	(III) Expenses Elsewhere Unprovided For.	
	(IV) Dividends on Outstanding Stocks.	
	(V) Miscellaneous Appropriations	(a) Amortization Elsewhere Unprovided For. (b) Appropriations to Reserves (c) Gifts to Controlled Companies. (d) Other Appropriations.
	(VI) Bad Debts Written Off.	
	(VII) Other Deductions from Surplus.	

intended to include those which convey water for long distances, without having any connections for services from the pumping station to the distributing storage reservoir or the distributing system mains. Accordingly only applying in the case of water works operating pumping stations, as in the case of a gravity system such mains would come under the classification of distribution or supply mains.

(6) Miscellaneous Properties, (III) Other Tangible Water Capital: All tangible property devoted to water operations not elsewhere provided for.

(7) Miscellaneous Construction Expenditures, (V) Miscellaneous Construction Expenses: Including all salaries and expenses of executives and general officers, salaries of clerks; all construction, equipment items of a special and incidental nature, and not properly chargeable to any other classification account.

EXPLANATION OF INCOME SCHEDULES GIVEN IN TABLE IV.

(1) Non-Operating Revenues, (I) Rents Accrued from Lease of Water Plant: All revenues from interests in water plant or equipment held by others under some form of lease whereby possession of such property is surrendered. (II) Miscellaneous Rent Revenues, (b) Other Rent Revenues: All such from any lease or contract the entire term of which does not exceed one year, or which is held at will, etc. (III) Interest Revenues (b) Interest from Other Sources: All interest accruing upon interest-bearing current assets, special deposits, etc., not provided for by classification elsewhere. (V) Miscellaneous Non-Operating Revenues: All non-operating revenues accruing to the company and not provided for elsewhere. (VI) Non-Operating Revenue Deductions, (d) Miscellaneous Non-Operating Expense: All non-operating expenses not provided for by other sub-accounts. (e) Non-Operating Taxes: All taxes accruing upon non-operating property and assignable to non-operating revenues.

(2) Income Deductions, (II) Other Interest Deductions: All interest accruing monthly or quarterly on receiver's certificates, and on all interest-bearing obligations. (IV) Miscellaneous Rent Deductions, (a) Joint Facility Rents: To include the rent, return, or profit, above depreciation and other expense of maintenance and operation, on plant or equipment operated by another company for the joint benefit of the accounting company and others, and under a joint arrangement for sharing the expenses. (b) Miscellaneous Rent Deductions: All miscellaneous rents payable, and not elsewhere provided for. (V) Miscellaneous Deductions from Income, (d) Other Contractual Deductions from Income: All contractual liabilities accruing from annual or periodic matters not elsewhere classified.

(VI) Amortization Deductions, (a) Amortization of Landed Capital: At the close of any fiscal period to include such proportion of the original money cost of landed capital as necessary to cover the extent of life expired during such period.

(3) Appropriation Accounts (II) Other Additions to Surplus: Providing for the crediting of all additions to surplus not otherwise classified. (V) Miscellaneous Appropriations, (d) Other Appropriations: All optional appropriations made, and not otherwise provided for. (VII) Other Deductions from Surplus: All deductions for surplus not elsewhere arranged for by sub-accounts.

TABLE V.—OPERATING REVENUE ACCOUNTS.

(1) Metered Private Service.
(2) Unmetered Private Service.
(3) Service to Other Water Supply Systems
(4) Municipal Water Service.
(5) Miscellaneous Water Service.
(6) Miscellaneous Operating Revenues.

EXPLANATION OF OPERATING REVENUE SCHEDULES GIVEN IN TABLE V.

(5) Miscellaneous Water Service. All revenues accruing for water furnished to builders or street paving contractors, and those using water for a limited period of time without permanent service connections; and for all other water service not elsewhere classified.

(6) Miscellaneous Operating Revenues: All revenues accruing for services rendered in connection with water operations, other than supplying water, such as fees for shutting off and turning on water, tapping mains, etc.

EXPLANATION OF OPERATING EXPENSE SCHEDULES GIVEN IN TABLE VI.

The accounts for operating expenses are arranged in three groups as follows: (1) Water Supply Expenses are those which represent the costs of materials and services other than those for repairs, and directly connected with collection, purification, pumping and distribution; (2) Maintenance Expenses are those which represent the cost of services and materials incident to the repair of properties, and the losses arising from the depreciation of same; (3) General and Miscellaneous Expenses cover all costs relating to the general management of the water works as a business enterprise, and those incident to the business as a whole not otherwise classified.

TABLE VI.—OPERATING EXPENSE ACCOUNTS.

(1) Water Supply Expenses..	(I) Collecting System Expenses	(a) Water Purchased. (b) Reservation and Drainage Area. (c) Impounding Dams and Reservoirs. (d) Lake and River Cribs. (e) Springs and Wells. (f) Infiltration Galleries and Tunnels. (g) Collecting Conduits and Reservoirs. (h) Gravity Intakes and Suction Mains. (i) Aqueducts and Supply Mains.
	(II) Purification System Expenses	(a) Sedimentation and Coagulation. (b) Softening and Iron Removal. (c) Filtration. (d) Chemical Treatment. (e) Clear Water Basins.
	(III) Pumping System Expenses	(a) Operating Labor. (b) Fuel. (c) Power Purchased. (d) Miscellaneous Supplies and Expenses.
	(IV) Distribution Expenses ..	(a) Transmission Mains and Accessories. (b) Storage Reservoirs, Tanks and Standpipes. (c) Distribution Mains and Accessories. (d) Service Pipes and Stops. (e) Meters, Meter Boxes and Vaults. (f) Fire Hydrants and Cisterns. (g) Fountains, Troughs and Other Equipment
(2) Maintenance Expenses...	(I) Repairs of Water Collecting System	
	(II) Repairs of Purification System.	
	(III) Repairs of Pumping System.	
	(IV) Repairs of Distribution System	(a) Transmission Mains and Accessories. (b) Storage Reservoirs, Tanks and Standpipes. (c) Distribution Mains and Accessories. (d) Service Pipes and Stops. (e) Meters, Meter Boxes and Vaults. (f) Fire Hydrants and Cisterns. (g) Fountains, Troughs and Other Equipment
	(V) General Repairs.	
	(VI) General Amortization.	
(3) General and Miscellaneous Expenses	(I) Administration Expenses ..	(a) General Officers. (b) General Office Clerks. (c) General Office Supplies and Expenses
	(II) Accounting and Commercial Expenses .	(a) Employees. (b) Office Supplies and Expenses
	(III) Business Promotion Expenses	
	(IV) Legal Expenses.	
	(V) Injuries and Damages	
	(VI) Insurance.	
	(VII) Relief Department and Pensions	
	(VIII) Water Franchise Requirements.	
	(IX) Store and Stable Expenses	(a) Store (b) Stable
	(X) Miscellaneous Adjustments (balance)	(a) Inventory Adjustments. (b) Discounts on Materials and Supplies (credit). (c) Duplicate Water Charges (credit) (d) Water Expenses Transferred (credit). (e) Joint Operating Expenses (credit)
	(XI) Miscellaneous General Expense	
	(XII) Taxes.	
	(XIII) Uncollectable Water Bills	

(1) Water Supply Expenses (III. Pumping System Expenses), (d) Miscellaneous Supplies and Expenses: All supplies and expenses incurred in operating the pumping plant, not otherwise classified, as water used in boilers, oil and waste, etc.

(2) Maintenance Expenses, (V) General Repairs: To include the cost of all labor and materials used in making repairs to general structures and general equipment, such as general offices, general storehouses, shops, etc. (VI) General Amortization: The amount

necessary to cover such wear and tear and obsolescence as have accrued during the period in the tangible fixed capital of the company, such period to be monthly or quarterly; also, such portion of the life of the intangible fixed capital as has expired or been consumed during the period, and any reserve amounts necessary to cover the cost of property destroyed by unusual casualties.

(3) General and Miscellaneous Expenses, (XI) Miscellaneous General Expenses: All miscellaneous general expenses not elsewhere classified, and of such unimportant character as not to warrant the opening of a specific account or accounts for same. (XIII) Uncollectible Water Bills: To include all bills rendered which, after reasonable period and efforts, have proved impracticable of collection. Upon charging the amount to this account, it should be credited to the account receivable in which it has been carried.

Discussion and Data on the Economic Location of Conduits.

Every conduit must change at certain points along its length from one type of construction to another, as from pipe line to lined canal, unless it traverses a country of uniform topography. Such changes arise from considerations of economy or safety. A conduit of any considerable length seldom consists wholly of canal section but changes from that type of construction to flumes, siphons, pipe lines, bridge flumes or tunnels, as the location conditions dictate. The points at which such changes are made are determined by economic considerations as well as by ground slope, by the nature of the material encountered in the excavation processes and by the other local conditions. Just as the comparative costs of deep cutting and tunneling are used to indicate when to pass from cut to tunnel in railroad location, so the transition from one type of conduit construction to another is based upon comparative costs.

In making various conduit locations Mr. C. E. Hickok, Assoc. M. Am. Soc. C. E., has evolved a diagram which gives the equivalent lengths, from an economic standpoint, of various types of conduits. The diagram and Mr. Hickok's discussion of it are here given as described by him in his paper entitled: A Study of Economic Conduit Location, as published in Vol. XXXIX Proc. Am. Soc. of C. E., pp. 2185-2190. When the locator comes to a point where he must decide whether to tunnel through a ridge or follow the grade around with a canal, he measures the length of the two possible routes, and, by an inspection of the diagram given herewith, reaches a decision as to the more economical of the two types of conduits. This not only saves time but, with a properly constructed diagram, assures a proper and complete comparison between the two alternatives as to first cost, depreciation, head-loss values, evaporation and seepage loss values, interest, taxes, inspection and repairs.

For purposes of illustration assume a case where the project under consideration is to be used for irrigation and hydro-electric purposes, and where the conduit has a capacity of 11.6 cu. ft. per second and a slope of one-tenth of 1 per cent. Four types of conduit are shown in Fig. 1.

It is obvious that for each foot saved in length of conduit there is a saving in head loss, as well as in evaporation and seepage losses. The value of this saving is ascertained in the following way, taking 1,000 ft. of conduit, for convenience in calculating:

Head Loss.—1,000 ft. of conduit dissipates 1 ft. head. With a discharge of 11.6 cu. ft. per second, and 77 per cent efficiency, the lost power is:

$$\frac{1 \times 11.6 \times 62.5 \times 0.77}{550} = 3.9 \text{ HP.} = 2.9 \text{ KW.},$$

less 10 per cent for transmission and transformer losses = 2.61 KW. at $55 = $143.50.

Evaporation Loss—Power Value.—Assuming an evaporation of 5 ft. per annum:

$$\frac{8 \times 1,000 \times 5.0}{43,560} = 0.915 \text{ acre-ft. per year}$$

= 0.0025 acre-ft. per 24 hours = 0.00125 cu. ft. per second., with a head of 1,500 ft.

$$\frac{0.00125 \times 1,500 \times 62.5 \times 0.77}{550} = 0.162 \text{ HP.} =$$

0.121 KW., less 10 per cent for transmission and transformer losses = 0.109 KW., at $55, = $6.

Seepage Loss—Power Value.—From tests made by Elwood Mead and B. A. Etcheverry, at the University of California, the writer concludes that the rate of percolation through a 3-in. canal lining under a head of 3.5 ft. is about 0.0043 cu. ft. per hour, or 0.103 cu. ft. per 24 hours.

$$\frac{8 \times 1,000 \times 0.103}{43,560} = 0.0188 \text{ acre-ft. per 24}$$

hours = 0.0094 cu. ft. per second.

$$\frac{0.0094 \times 1,500 \times 62.5 \times 0.77}{550} = 1.23 \text{ HP.} =$$

0.92 KW., less 10 per cent = 0.828 KW.; 0.828 KW. at $55 = $45.54.

Total annual power loss, $195.04.
Capitalized at 10 per cent, $1,950.40
Or per foot, $1.95.

Evaporation Loss—Irrigation Value.—0.0025 acre-ft. in 24 hours (from the foregoing) = 0.00125 cu. ft. per second = 0.0625 miner's inch. Assume 25 per cent loss before delivery to consumer.

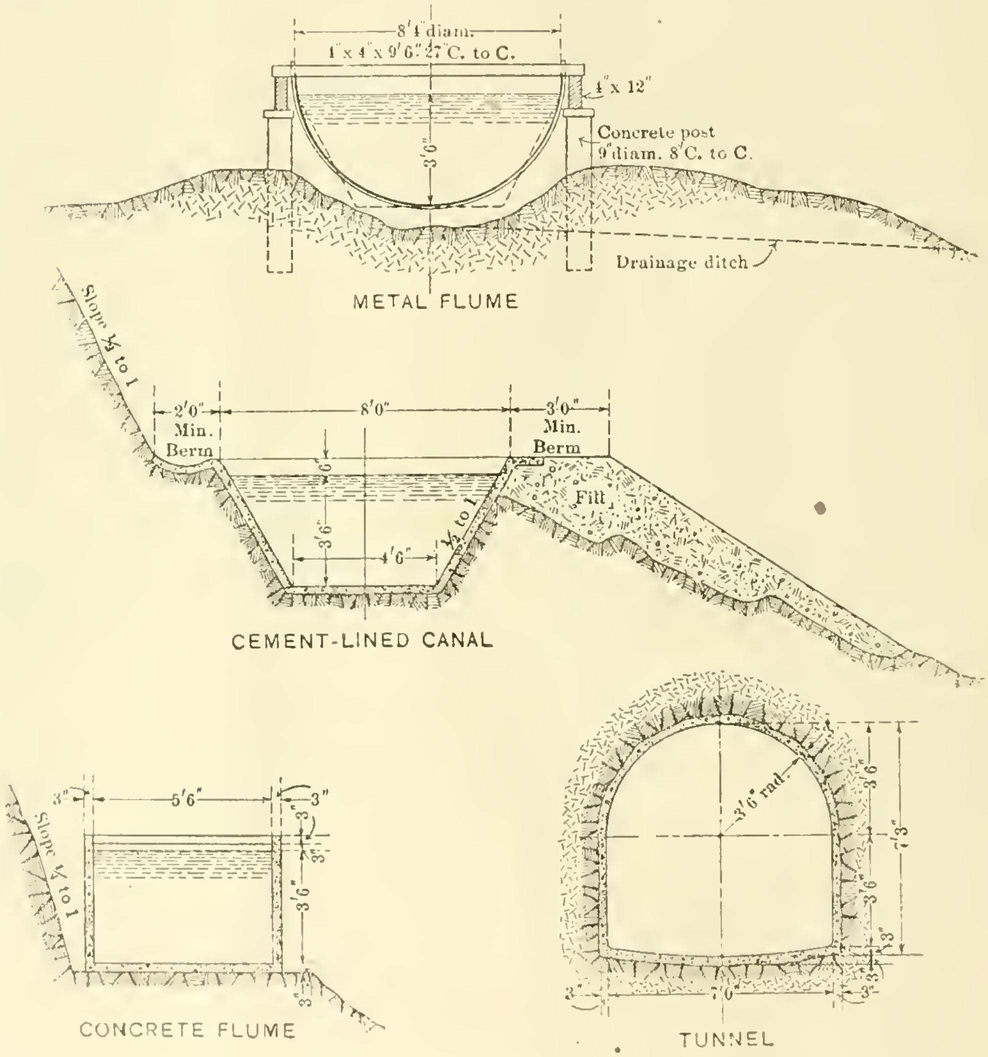
0.047 miner's inch at 40 cts. per miner's inch per day = per annum, $6.86.

Seepage Loss—Irrigation Value.—0.0188 acre-ft. per 24 hours (from the foregoing) = 0.0094 cu. ft. per second = 0.47 miner's inch, less 25 per cent loss = 0.353 miner's inch at 40 cts. per miner's inch per day = per annum, $51.64.

Total annual irrigation loss, $58.50.
Capitalized at 10 per cent, $585.00.
Or per foot, $0.585.

Summary.—Power loss per foot, $1.95; irrigation loss per foot, 0.585; total loss per foot, $2.535.

The first cost and the annual charges of each type of conduit are next computed. The annual charges are taken as consisting of the following items: Interest, depreciation, taxes, inspection, and repairs. The annual charges of each conduit are capitalized at 10 per cent and added to its first cost, which gives a figure having a real comparative value. For instance, we obtain the comparison between a lined canal and a concrete-lined tunnel as follows:



METAL FLUME

CEMENT-LINED CANAL

CONCRETE FLUME

TUNNEL

Fig. 1. Economic Conduit Location—Cross Sections of Four Types of Conduits.

CONCRETE LINED CANAL.

First Cost—Per Foot.—Excavation, 2 cu. yds. at 36 cts. = $0.72; concrete, 4.25 cu. ft. at $10.20 per cu. yd. = $1.57. Total = $2.29.

Annual Charge.—Interest at 10 per cent, $0.23; depreciation, at 2 per cent, $0.046; taxes, $0.019; inspection, $.01; repairs, $0.02; total, $0.325; at 10 per cent, $3.25; total, $5.54

CONCRETE-LINED TUNNEL.

Excavation, 2.25 cu. yds. at $5.50 = $12.40; concrete and forms, $1.10; total, $16.50;

Annual Charge.—Interest at 10 per cent, $1.65; depreciation, at 1 per cent, $0.165; taxes, $0.137; inspection, $0.01; repairs, $0.02; total, $1.982; at 10 per cent, $19.82; total, $36.32.

It is evident, if we shorten the conduit by building the tunnel, that the first cost and the capitalized annual cost of the tunnel can exceed the first cost and the capitalized annual cost of the canal by an amount equal to the length of conduit saved multiplied by the loss value per foot of conduit. This is shown by the equation:

$Y\ (C_y + A_y) = X\ (C_x + A_x) + (x - y)\ V$

where

X = linear feet of canal.
Y = linear feet of tunnel,
C_x = estimated cost per foot of canal.
A_x = estimated annual charges per foot of canal capitalized at 10 per cent,
C_y = estimated cost per foot of tunnel.
A_y = estimated annual charges per foot of tunnel capitalized at 10 per cent, and
V = value of losses per foot of conduit.

In the case of a tunnel, the evaporation will be considerably lessened, thereby effecting an additional saving. If entirely eliminated, this saving would amount to 12.8 cts. per foot, as shown above. This was reduced to 10 cts. and the first cost of tunnel credited with that amount. Inserting the proper values in the equation:

$$Y\ (16.40 + 19.82) = X\ (2.29 + 3.25) + (x - y)\ 2.53$$

$Y = 0.208\ X$, the equation of a straight line.

In the same way, two types of conduit can be compared and the resulting straight-line equation obtained. The diagram, Fig. 2, which is self-explanatory, shows the results. The following formula and assumptions were used in constructing Fig. 2:

GENERAL FORMULA.

$$y(C_y) = xC_x + (x - y)\ V + x\ (A_x) - y(A_y)$$

C_y = Estimated cost per foot of conduit above line.
A_y = Estimated annual expense per foot of conduit above line capitalized at 10 per cent.
C_x = Estimated cost per foot of conduit below line.
A_x = Estimated annual expense per foot of conduit below line capitalized at 10 per cent.
V = Estimated value of one foot of canal for power and irrigation purposes = \$2.53.

Tunnel is credited with 10 cts. per foot for saving in evaporation.

VALUES USED.

	Original cost per ft.	Annual expenses per ft.
Tunnel	\$16.50	\$1.98
Concrete-lined canal	2.30	0.325
Concrete flume	4.28	0.579
48-in. steel siphon, 100-ft. head	5.88	0.961
48-in. steel siphon, 200-ft. head	7.46	1.211
48-in. steel siphon, 300-ft. head	10.64	1.714
Steel flume	5.58	0.887

Steel siphons are credited with \$1.10 per ft. for saving in evaporation and seepage.

In the case where a siphon crossing a gulch is compared with a canal or flume passing around the head of the gulch, the cost of the siphon is credited with the saving in evaporation and seepage throughout its length, which in this case amounts to \$1.10 per foot.

The writer realizes that such a diagram cannot be relied on entirely in the location of a conduit, for there are local conditions on every piece of work which must be taken into account.

Some Aspects of Water Supply Engineering in Communities of 300 to 3,000 Population.

In discussing some of the aspects of and difficulties encountered in the water works business in cities and towns of the third and fourth classes, Mr. Morton Macartney, city engineer of Spokane, Wash., stated that in his opinion the chief interest in the subject lies in the cities of from 300 to 3,000 population. Mr Macartney's paper, which was presented before the Washington League of Municipalities, related primarily to conditions existing in that state, where fourth-class cities range from 300 to 1,500 population and third-class cities from 1,500 to 10,000 population. The conditions in cities of these classes in Washington, however, are so diversified and present such a wide field that any problems arising in such towns may fairly be considered typical of conditions in towns of like size throughout the country.

The field is so broad that it includes everything connected with water supply. For the communities of the size in question, nearly every known source of water supply has been utilized in Washington from a mountain stream or spring, piped and conveyed by gravity to the consumer, to a deep well with the most complicated pumping, storage and distribution system, and all types apply to cities of these classes. The large range of sizes in towns of the third and fourth class as applied to water supply is in itself no small item. Of the two classes under discussion, the fourth class presents the more difficult problem, although the conditions surrounding each individual water supply system will be simpler than those of the third class, and our discussion will primarily deal with towns of the fourth class. The difficulties confronted in the two classes of towns are more matters of degree than of character, and while the suggestions made are particularly applicable to towns of the fourth class, I think they will apply to third class cities, only in a slightly less degree.

There are about 112 towns and cities of the fourth class in Washington, and of these there are 39 towns having a population of from 300 to 500; 46 towns having a population of from 500 to 1,000 and 27 towns having a population of from 1,000 to 1,500. There are only 39 towns and cities of the third class in Washington, of which 20 have from 1,500 to 3,000 inhabitants. Ten cities have from 3,000 to 5,000, and only nine cities have a population of from 5,000 to 10,000. The chief interest in this subject, it seems to me, lies in the cities of from 300 to 3,000 people.

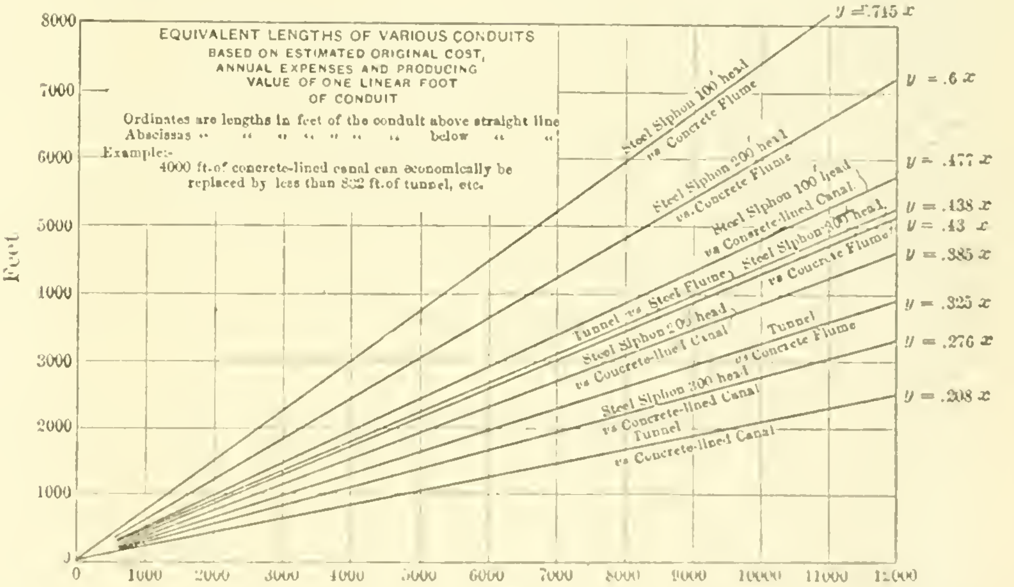


Fig. 2. Economic Conduit Location—Equivalent Lengths of Various Types of Conduits, All Costs Considered.

When a community reaches the point of having a population of about 300, it usually begins to realize that it is a community having common interests and common desires. Naturally, the first serious problem that comes to it is that of water supply.

Few of the towns of Washington having a population of less than 400 have a public water supply, but in almost every one of these communities there is more or less discussion about this question. The writer sent out letters to every town and city of the third and fourth classes in the state, asking for detailed information regarding their water supplies. In all, 60 replies were received, and the information given was in the main satisfactory, although in a great many instances the information was meager, and from the manner in which a great many of the questions were answered, was not considered entirely reliable. In 17 replies to [illegible] questions sent to towns of less than 400 people, [illegible] towns only reported water systems in operation, [illegible] under construction and [illegible] contemplating such construction.

To the latter class, those beginning to plan for a water system, a serious question presents itself. Its citizens usually do not appreciate the necessity of securing technical advice in these matters, advice which would safeguard the community from the usual blunders in starting work of this character. Their funds are very limited and they will not allow proper preliminary expenditure, and the many makeshifts that exist are the result. The state of Washington, however, has really been rather fortunate in this matter, for it does not possess to as large a degree the great number of inefficient, badly planned and badly constructed water systems as many other older states possess. Investigations by competent men are usually not cheap processes, although usually the most economical in the end, and so many instances of poor advice and poor plans exist, which have come from sources supposed to be competent, that our citizens in the small places are very reluctant to authorize the preliminary expenditure for investigation.

One hears much complaint among engineers against the practice of technical men connected with the state institutions undertaking the planning and execution of works of this character, claiming that they are being deprived of a legitimate line of practice, a source of income for them to pay the very taxes which support the several state institutions. A discussion of this phase of the matter would raise the entire question of the seeming paternal element that is fast creeping into our state and also our national government, and only one suggestion will be offered in this matter, and that from the point of view of the citizens of the small town. The latter is almost helpless in selecting an adviser from the list of technical talent available, men of all grades of ability, from an ordinary plumber to a highly trained and experienced hydraulic engineer. There were in Spokane about a year ago some 60 men advertising themselves to be experts in water works construction, most of these having references which would pass the inspection of an ordinary layman. To my personal knowledge 10 of these men have never made an installation of the character in question; as a consequence the mayor, aldermen or commissioners of the small town, conscientious and desirous to get the most for the money his town can spend, will turn to the man whom he believes to be competent because of the very connection with the various state institutions. He looks upon, rightly or wrongly, connection with a state institution as sufficient endorsement for the ability and integrity of the man.

There are two ways of overcoming this difficulty—one by examining and licensing engineers who are able to execute plans for this type of work and requiring the towns, particularly the fourth class, to employ such a licensed engineer, or, second, the approval of all plans for small water works systems by a competent state official or a board of such officials. The fees for technical advice for works of this character from cities of the fourth class, and even the third class, are so small that they attract only the poorer and inexperienced man. In many instances this condition results in the only advice being asked is that of an expert employed by a manufacturing concern or a supply house. His chief aim many times is to sell certain classes of machinery or supplies rather than furnishing a well-designed and efficient plant. Instances of this character are too numerous, not only in Washington, but over the entire country.

To try to outline any definite set of rules to apply to cities of this class for the planning and construction of water works would be absurd, on account of the varying conditions and sizes, amount of probable consumption, possible sources and actual physical conditions of the town itself, and one set of rules proper for one community would be of no value to another.

The available data concerning towns of the fourth class, and even of the third class, is very meager and difficult to collect. In 10 replies from cities having a population of from 500 to 1,000, only one city could tell the amount of water used in 24 hours. Most of these replies said there was no way of determining this, although many reported that they were pumping water from wells and streams. While the consumption per capita of towns of 500 to 1,000 will probably be a little less than in those from 1,000 to 5,000, the difference will be slight.

In towns of 1,000 to 5,000 (of the 24 towns replying) the average daily consumption per capita was a little less than 200 gals. In these replies, almost without exception, the irrigating systems were separate from the domestic water supply, other than the little lawn irrigation that is done with hose attached to taps on the water mains. Of towns larger than 5,000 only one reply was received, and that indicated a daily consumption per capita of 110 gals. The consumption indicated in all replies was between widely varying limits, but when averaged showed a per capita consumption usually followed in well-designed pumping plants. The sources of water supply are so numerous that to give any intelligent idea of the same would be to enumerate each one and describe its peculiar difference from the others. In general, the replies received indicate that of the 60 towns five obtain their water from mountain streams, 12 from mountain springs, 12 from wells (no indication was made as to whether these wells were deep or shallow), three from lakes and 28 did not indicate the source of their water supply.

Municipal ownership in these classes of towns and cities is predominant. Only 10 of the 60 towns replying indicated private ownership, and of these three were negotiating for the purchase of these water supplies. It is a peculiar coincidence that the replies regarding rates charged would indicate that the lowest rate charged and also the highest rate charged were reported from cities having private ownership. While the writer believes that the towns of this character should always own their own water systems, the data received does not offer any true basis for comparing the merits of public and private ownership.

I have mentioned in the foregoing the great variety of conditions of water supplies, and the one element offering probably the largest limits is that of the rates charged. Most of the 60 towns reported the use of meters and gave rates charged on a meter basis. There were, however, several towns which reported a flat rate. Where flat rates were reported the amount charged did not vary widely. The lowest flat rate reported was 95 cts. per month for an ordinary residence having modern facilities, such as bath and lavatories. The highest rate charged for the same class of service was $1.50 per month. Without exception, where this flat rate was charged, a difference is made in the house having modern facilities and the one having none. A sprinkling rate for the irrigation season is usually charged where flat rates are used. The average of the eight towns reporting a flat rate was $1.20 per month for a modern house. Where meter rates are used, such rates are hard to reduce to terms comparable. Almost invariably a minimum rate is charged, although there are exceptions to this. Two of the towns charge a rate per 1,000 gals. whether 1,000 or 10,000 gals. are used, but the rule is that a certain rate is charged for the first 3,000 to 6,000 gals. Where a flat rate is charged for 1,000 gals. without a minimum rate, the rates are exceptionally low and indicate a very economical installation and low operating cost. One town reports a rate of 20 cts. per 1,000 gals. for any quantity of water used. Another, 30 cts. per 1,000 gals. for the first 25,000 gals. a reduction of 5 cts. being made for every 10,000 gals. used above this amount, until a rate of 10 cts. per 1,000 gals. is reached. This rate, however, includes ordinary irrigation charges. The rates charged are shown in Table I, and while all of the cities reporting are not shown, enough of these are shown to set forth the typical conditions throughout the state.

TABLE I.—TYPICAL METER RATES FOR WATER IN SMALL TOWNS IN WASHINGTON.

Gals.— 1,000.	2,000.	3,000.	4,000.	5,000.	6,000.	7,000.	8,000.
$2.80	$3.35	$5.40					
←——1.25——→				5c per M.			
←——1.00——→				15c per M.			
←——1.50——→			30c per M.				
←——1.05——→			5¼c per M.				
$4 25	18c per M.						
20c	...	...	...	...	...	...	
←——1.00——→					20c per M.		
←——1.50——→					7½c per M.		
←——1.00——→			10c per M.				
30c per M. 1st 25,000. Reduction of 5c in rate every 10 M. to 10c.							
←——1.00——→				1.00 per M.			
←——2.00——→					25c per M.		
←——1.50——→			1.50 per M.				
←——1.00——→						8c per M.	
←——3.00——→					14c per M.		
1.00	25c	15c	...	...	...		
40c	...	...	...	...	30c	25c	
1.50	40c per M.						

Most small places pay little or no attention to the fire-protection end of their water supply, other than the installation of fire hydrants on the system. Adequate storage capacity for fire protection is seldom provided, and, when not provided, there is usually no auxiliary pumping arrangement made to cover the same condition. Where the total consumption of water is 100,000 gals. (the average of the towns of 1,500 people) there should either be a storage of twice the amount of the total daily consumption or a total pumping capacity of sufficient size to completely cover fire protection. For a town of this size such a capacity is one that will throw three fire streams at the rated normal head of 125 lbs. The exact method of reaching this result is, of course, a matter to be determined after a careful study of local conditions, such as the cost of electrical energy or other pumping power, cost of constructing reservoirs and the ability to secure rates for low-load factors where electric energy is used. Where electrical energy is available an auxiliary system may often be installed and be in readiness for fire hazard, upon the payment of a small "readiness to serve" charge.

The distribution systems are usually designed in the cheapest manner to serve the first or primary demands of the town, and no consideration is given to important factors, such as the delivery of water without undue loss of pressure, flexibility of control and operation and the possibility of future extension.

Water supply in cities of the third and fourth class, like nearly all questions pertaining to community life, are ones to be determined by trained men, regardless of personal interests; men who have not only the present but the future welfare of the community in consideration. The best advice, whether technical, legal or financial, is none too good for a community starting out on a venture such as the consideration of a water system, and when an adviser offers his services to a community for this class of work it should be given the closest scrutiny to the end that not only his training and experience should be the best, but that his character and integrity are above question. With such an adviser, few mistakes are made and good results are almost invariably secured.

Organization and Procedure of and Instructions Issued to Water Waste Prevention Force in Philadelphia—Results.

The systematic attempt now being made to curtail the waste of water in Philadelphia was begun in April, 1912, by starting a thorough house-to-house inspection for leaking fixtures. In August of that year pitometer surveys were begun. Mr. John S. Ely, assistant engineer in charge of the waste prevention work, described the organization of the force engaged on that work in a discussion on water waste prevention methods before the New England Water Works Association. Mr. Ely also described the procedure followed in carrying on the work, described the apparatus used and enumerated the general instructions given to the inspectors. The following information is taken from his discussion:

For convenience in house-to-house inspection, the city was divided into 12 sections, letter A, B, C, etc., and the blocks in each section were numbered consecutively. Fifty men were employed, at $2 per day, with an experienced district inspector as foreman, who laid out the work for his men each morning, assigning a certain block to each man. These inspectors found a great many leaks, and the results of their inspection began to be apparent immediately in reduced pumpage.

Use of Aquaphones.—The corps has since been weeded out and reduced to 30, the pay increased to $2.50 per day, and the men provided with aquaphones and taught how to use them. These aquaphones are invaluable in locating leaks in underground service and yard hydrant pipes. By their use about 1,500 such leaking underground pipes have been found.

The aquaphones are also valuable in expediting the work of inspection, for the inspector, by listening on a single spigot, can tell at once whether any water is running in the house, and if there is none, can proceed to the next house, thus saving the time and the inconvenience to the householder of a complete inspection.

GENERAL INSTRUCTIONS.

The men wear badges and carry cards for identification, and the following general instructions are issued to them:

1. Inspectors must wear their badges on the outside of their coats in plain sight.
2. They must at all times be polite and courteous in their manner, and must cause householders as little inconvenience as possible.
3. They must ask permission of the householder before entering private rooms or bathrooms, and avoid doing anything which might be misconstrued on the part of the householder.
4. They will not waste time over small and trivial leaks. In leaks smaller than ¼-in. stream, it is sufficient to call the tenant's attention to them, and no written notice to repair need be made out. It is of much more importance to spend the time in finding large leaks.
5. They will in all cases carefully examine the service pipe from the street, using an aquaphone, and in cases where there seems to be a leak in the service, it shall be followed up until located and shall be reported to the foreman at the first opportunity.
6. In cases where leaks are found of ¼-in. stream or more, the regular printed notice to repair will be carefully made out and left with the tenant, the duplicate copy being kept and given later to the foreman.
7. Inspectors will spend only enough time in each house to walk rapidly through and note

the condition of plumbing. Five or ten minutes should be sufficient in all cases.

8. In cases where leaks of ¼-in. or more are found in rented houses, the name and address of the owner or agent should be taken and written on the duplicate slip.

9. Inspectors shall keep steadily at work during the whole day and endeavor to find as many large leaks as possible. The city pays you for your time and is entitled to it.

10. As inspectors are intrusted with discretionary duties, they will be required to display good judgment on all occasions, and lapses from a high standard of conduct will be dealt with accordingly.

A second inspection of those houses where notice to repair has been left is made about one week later. If the leak is still unrepaired, at the second inspection, a "shut-off" man is sent in a few days, and, finding the leak still not repaired, he shuts off the water at the curb.

Two clerks keep the records of the inspections and lay out the work for the second inspections and "shut-off" men.

The work of this corps is best shown by a summary of the first six months of 1913, as follows:

Number of buildings inspected	113,808	
Number of buildings showing waste	20,780	(18 per cent)
Number of buildings repaired	17,371	
Number of buildings shut off	2,121	(many vacant)
Fixtures repaired, toilet flush tanks	12,432	
Fixtures repaired, spigots	6,663	
Fixtures repaired, yard hydrants	1,286	
Fixtures repaired, horse troughs	36	
Fixtures repaired, service and hydrant pipes	2,367	

COST OF HOUSE-TO HOUSE INSPECTION AND REPAIRS.

	Cost.	Cost per building inspected.
Labor	$7,908.21	$0.0695
Material	100.40	....
Transportation	189.10	....
Total	$8,197.71	$0.072

The ordinary flush tank of cheap make is responsible for a very large percentage of the total waste. It is often concealed and a good sized flow may run constantly with very little sound.

In August, 1912, the bureau purchased six Simplex pitometer sets, including recorders. The recorders are of a recently perfected type, giving a 24-hour record of the velocity of flow in red ink on a circular chart, driven by clockwork. They were first used to check the output from the various pumping stations and filter plants. They are now being used to measure the consumption in various districts of the city, a district being isolated in the usual manner by lines of closed valves and the flow into it measured on one or more large feed mains.

In addition, a number of pitometer rods were purchased for use with a U-tube in making night block surveys. For this purpose the apparatus is set up in a shelter box over a tap in the street main at some convenient point. The adjacent territory is then cut in, a block at a time, by closing the valve at one end and feeding past the pitometer. An observer notes the deflections in the U-tube and the time of each deflection. Thus, the night rate of flow into each block is obtained (the work always being done between 11 p. m. and 5 a. m.). A report of these rates is turned in the following morning and house-to-house inspectors are sent through all the blocks which show a high rate until it is satisfactorily accounted for.

In the six square miles of territory examined in this way, only two leaks have been found in the street mains, these being small joint leaks. In all other cases, the leakage has been accounted for in broken service pipes or house plumbing.

A recent improvement in the method of making these night surveys has been made in the following manner: A short section of 6-in. flange pipe has been mounted on a light covered wagon, the pipe having 3-in. hose connections at either end, and a 1-in. ferrule or pitometer connection in the center. The U-tube is fastened up on the inside of the wagon cover. The block to be examined is shut off at each end, and fed through a fire hydrant from another hydrant on a live block, the wagon being of course stationed between the two hydrants, thus measuring any leakage there may exist in the block.

This method has the following advantages:

1. It permits quicker work because there are fewer valves to handle. The average number of blocks covered in one night is about 20 with the wagon and 15 by the old method.

2. It is cheaper and does away with the necessity of tapping the main for a pitometer connection.

3. It can much more readily be taken to any part of the city, where surveys are desired.

It has the disadvantages of being less accurate in measuring the amount of leakage, giving rates about 30 per cent less than the old methods, due probably to loss of pressure in passing through the hose. An exact value of the leakage is not, however, necessary in rapid surveys of this character.

These surveys, when followed up by a through and efficient corps of house-to-house inspectors, are probably the quickest method of reducing waste in any given territory. It is not necessary for the pitometer corps to stop and examine each house and service individually, but only to find the block night rate. Likewise, it is not necessary for the corps of inspectors to examine those blocks where there has been shown to be no leakage. Each corps, therefore, reduces the work of the other. If at the same time, the district in which the work is being done is isolated and the flow into it being measured by recorders, the results of the work can be readily followed. The effects of this work are soon felt in the neighborhood in which it is being done in increased pressures.

The net result of the work done so far has been to reduce the rate of consumption in West Philadelphia from 115 gals. to 125 gals. per capita per day, or about 14 per cent.

The total pumpage for the whole city in May, June, and July, 1913, averaged 301.638 m.g.d., while for the same three months in 1912 it averaged 315.9 m.g.d., a saving of 4½ percent, although only about one-tenth of the area has yet been covered.

Roads and Streets

Methods and Costs of Constructing the Byberry and Bensalem Service Test Road, Philadelphia, Pa.

In the past few years a number of experimental roadways have been constructed in different parts of the United States. A new road which is claimed to be in no sense an experimental road, but distinctly a service test roadway embracing practically all of the present day standard methods of construction and materials used for roads designed for both motor and animal drawn traffic has recently been completed in the city of Philadelphia. The accumulation of data to be used as a future guide in connection with country road construction in Philadelphia made the construction of such a road quite desirable. The results of experience gained through, and the data collected from, the various experimental roads were taken into account in the selection of methods and materials used in constructing the service test roadway which forms part of the Byberry and Bensalem Turnpike, a direct road to Trenton, Newark, Jersey City and New York.

The construction work was commenced on Sept. 28, 1912, and was suspended during the winter season, when the weather was too cold for work of that character. Operations were resumed again on April 11, 1913, and the roadway was completed Sept. 13, 1913. On account of lack of transportation facilities and the consequent scarcity of labor, the work was carried on with difficulty. With few exceptions, all of the labor employed came from the built-up portion of the city and their transportation was paid by the contractors. This expense was approximately 7 per cent of the total labor cost. A building was provided for the men to live in, but, although the work was some distance from transportation facilities, most of the men insisted in going to their homes in the city. Although the laborers were paid for a ten-hour day the work could not be prosecuted for ten hours every day on account of the necessity of allowing the men time to make connections with trains. The materials used were shipped to Busleton, Byberry, Somerton and Trevose stations on the Philadelphia & Reading Railway. The average length of haul from the railroad to the work was 1½ miles, which increased the cost of the work.

The roadway comprises 26 sections consisting of different types of construction. The work was done by contract. The methods and detailed costs of construction form the major portion of a report to the Director of Public Works on the construction of the Service Test road by Mr. William H. Connell, Chief Bureau of Highways and Street Cleaning, from which the information here given concerning the construction of the various sections, is taken.

SECTION 1, STATION 0 TO 37—BITUMINOUS PAVEMENT, MIXING METHOD, AMIESITE.

Foundation Course.—The foundation course was originally old waterbound macadam with a telford base. After cleaning and spiking the old macadam, 1½ in. trap rock was spread and rolled to bring the surface to a grade 2 ins. below the finished grade. Just enough screenings were added to form a coarse grainy base and the road was rolled and puddled until thoroughly cemented. A 12-ton 3-wheel roller was used.

Surface Course.—The paving mixture was shipped from the plant of the Amiesite Company, and consisted of clean crushed trap rock passing a 1½-in. screen treated with a liquifier, and mixed with asphaltic cement, after which oxide of lime was added. The paving mixture consisted of the following proportions:

	Per cent
Mineral aggregate	92
Asphalt	5
Lime	1½
Liquifier	1½

The material was spread and rolled to a depth of 2 ins. and after rolling once over, a final course of friable filler, consisting of a mineral aggregate of trap rock passing a ⅛ in. ring and treated and mixed in the same proportion as above described, was then spread and rolled with a 12 ton roller.

Surface Finish.—Upon the wearing surface a thin layer of clean trap rock screenings was spread and rolled.

Cost—

Item	Cost per sq. yd.
Preparation of foundation course	$0.1410
Surface course	0.6226
Surface finish	0.0075
Trimming shoulders	0.0062
	$0.7773

Detailed Cost Preparation and Foundation Course:

Materials— Item:	Cost per sq. yd.
1½-in. trap rock, 71 cu. yds., at $1.75	$0.0184
Trap rock screenings, [illegible] cu. yds., at $1.75	0.0130

Labor—	
Item:	
Scarifying	$0.0269
Shaping	0.0108
Hauling trap rock	0.0196
Spreading trap rock and screenings	0.0036
Rolling	0.0287
6,758 sq. yds. at	$0.1210
Wearing Surface—	
Materials—	Cost per sq. yd.
Item:	
Amiesite No. 1, 6,316 tons, at $3.928	$0.3671
Amiesite No. 2, 189.2 tons at $3.992	0.1118
Labor—	
Item:	
Unloading Amiesite	$0.0331
Hauling Amiesite	0.0572
Spreading Amiesite	0.0360
Rolling Amiesite	0.0171
6,758 sq. yds. at	$0.6226
Surface Finish—	
Materials—	Cost per sq. yd.
Item.	
Trap rock screenings, 8.89 cu. yds., at $1.755	$0.0023
Labor—	
Item:	
Hauling screenings	0.0015
Spreading	0.0037
6,758 sq. yds. at	$0.0075
	Cost per sq. yd.
Trimming Shoulders—	
Labor, 6,758 sq. yds. at	$0.0062

SECTION 2, STATION 37 TO 47—BITUMINOUS PAVEMENT, MIXING METHOD, TOPEKA.

Foundation Course.—The foundation course was originally old waterbound macadam with a telford base. After cleaning and spiking the old macadam, additional 1½-in. stone was added to give not less than 4 ins. of stone on the top of the telford foundation. Just enough screenings were added to form a coarse grainy base and the road was rolled and puddled until thoroughly cemented. A 12-ton 3-wheel roller was used.

Surface Course or Paving Mixture.—The paving mixture consisted of the following mineral aggregate: Trap rock ships, screenings and sand, within the limits of the following proportions:

	Per cent.
Passing 200 mesh screen	5 to 11
Passing 40 mesh screen	18 to 30
Passing 10 mesh screen	25 to 55
Passing 4 mesh screen	8 to 22
Passing 2 mesh screen	less than 10 per cent
Bitumen	10 per cent

The asphaltic cement used was Pioneer Road Asphalt and was heated in a tar kettle. The mineral aggregate was heated in a portable Rapid Heater Mixer; the hot asphaltic cement was added and mixed with the mineral aggregate.

The paving mixture was laid at a temperature of 250° to 350° and rolled to a finished depth of 2 ins., with a 6-ton tandem roller.

Seal Coat.—From station 42 to 47 a seal coat of ⅓ gal. per square yard of Pioneer Road Surface Asphalt was applied at a temperature of 350° and covered with clean trap rock chips passing ¾-in. screen and rolled.

Cost—	Cost per sq. yd.
Item:	
Preparation of base course	$0.3593
Wearing surface	1.0784
Sand covering	0.0077
Trimming shoulders	0.0792
	$1.5246
Seal coat	0.0872
Cost, including seal coat	$1.6118

Detailed Cost, Preparation of Base Course—	
Materials—	Cost per sq. yd.
Item:	
1½-in. trap rock, 174.35 cu. yds., at $2.3121	$0.1926
Trap rock screenings, 35.34 cu. yds., at $2.3121	0.0391
Labor—	
Item:	
Scarifying	$0.0096
Shaping	0.0417
Spreading trap rock	0.0396
Rolling and watering	0.0339
2,092.7 sq. yds. at	$0.3593
Wearing Surface—	
Materials—	Cost per sq. yd.
Item:	
Trap rock, 113.81 cu. yds., at $3.5914	$0.1957
Asphaltic cement, 46,920 lbs., at $0.013625	0.3048
Sand, 14.11 cu. yds., at $2.312	0.0156
Sand, 68.1 cu. yds., at $1.9785	0.0644
Kerosene, 9 bbls., at $6.22	0.0268
Coal, 7 tons, at $5.00	0.0167
Forms, etc., 100 ft. B. M., at $8.50	0.0001
Labor—	Cost per sq. yd.
Item:	
Mixing	$0.2437
Applying bituminous materials	0.1028
Rolling	0.1077
2092.7 sq. yds. at	$1.0784
	Cost per sq. yd.
Trimming Shoulders—	
Labor, 2092.7 sq. yds. at	$0.0792
	Cost per sq. yd.
Sand Covering—	
Materials—Sand, 4.18 cu. yds. at $1.9785	$0.0069
Labor—1203.8 sq. yds. at	0.0077
Seal Coat—	
Materials—	Cost per sq. yd.
Item:	
½ in. trap rock, 7.41 cu. yds., at $3.5944	$0.0300
Asphaltic cement, 2,360 lbs., at $0.013625	0.0362
Labor—	
Item:	
Sweeping	$0.0007
Heating asphaltic cement	0.0011
Applying asphaltic cement	0.0065
Spreading chips	0.0055
Rolling	0.0072
888.9 sq. yds. at	$0.0872

SECTION 3, STATION 47 TO 50—FIVE-INCH CONCRETE PAVEMENT WITH BITUMINOUS TOP.

Foundation Course.—The foundation course was originally old waterbound macadam with a telford base.

The surface was graded to within 5½ ins. of the finished grade and additional material was added where required. It was then thoroughly rolled.

Concrete Pavement.—A 5-in. concrete base, proportioned 1-3-6, was laid upon the prepared subgrade without joints. The concrete was mixed in a portable rapid mixer, and the surface was finished to grade with a template and trowels and swept with a stiff broom when partially set.

Bituminous Top.—A bituminous top of ½ gal. per square yard of Pioneer Road Surface Asphalt was applied at a temperature of 350° and covered with clean trap rock chips passing ⅜-in. screen and rolled with a 6-ton tandem roller.

Cost—	Cost per sq. yd.
Item—	
Preparation of base course	$0.0571
Concrete pavement	0.9500
Bituminous top	0.1261
Trimming shoulders	0.0130
	$1.1462
Concrete pavement per cu. yd.	$6.84

Detailed Cost—Preparation of Base Course.	Cost per sq. yd.
Material—	
Item:	
1½-in. trap rock, 4.73 cu. yds., at $2.312	$0.0205
Labor—	
Item:	
Scarifying	$0.0132
Shaping	0.0234
533.3 sq. yds. at	$0.0571
Concrete Pavement—	
Materials—	Cost per sq. yd.
Item:	
1½-in. trap rock, 72.75 cu. yds., at $2.312	$0.3154
Sand, 36.4 cu. yds., at $1.8432	0.1258
Forms, etc., 75-ft. B. M., at $8.50	0.0012
Portland cement, 90.75 bbls., at $1.85	0.3148
Labor	0.1928
533.3 sq. yds. at	$0.9500
Bituminous Top—	
Materials—	Cost per sq. yd.
Item:	
⅛-in. trap rock, 4.44 cu. yds., at $3.5914	$0.0299
Pioneer road surface asphalt, 2,260 lbs., at $0.012395	0.0525
Labor—	
Item:	
Sweeping	$0.0045
Heating bituminous materials	0.0108
Applying bituminous materials	0.0081
Chipping	0.0098
Rolling	0.0105
533.3 sq. yds. at	$0.1261
	Cost per sq. yd.
Trimming Shoulders—	
Labor—533.3 sq. yds. at	$0.0126

SECTION 4, STATION 50 TO 52 + 50—VITRIFIED BLOCK PAVEMENT ON 4-IN. CONCRETE BASE.

Foundation Course.—The foundation course was originally old waterbound macadam with a telford base.

The surface was graded to within 9½ ins. of the finished grade and additional material was added where required. It was then thoroughly rolled.

Concrete Base and Vitrified Blocks.—A 4-in. concrete base with concrete curbs, proportioned 1-3-6, was laid upon the prepared subgrade. The concrete was mixed in a Ransome No. 2 Concrete Mixer, equipped with a charging device.

The sand cushion was spread and rolled with an iron hand roller and shaped with a template. After laying, the vitrified blocks were rolled with a 6-ton tandem roller, and grouted to within ½ in. of the surface with cement grout, proportioned 1-1, and covered with sand.

Cost—	Cost per sq. yd.
Item:	
Preparation of foundation course	$0.1230
Concrete base	0.7366
Concrete curb	0.1196
Sand cushion	0.1378
Vitrified blocks	1.2563
Laying vitrified blocks	0.1095
Grouting and covering	0.1067
Trimming shoulders	0.0636
	$2.6531
Concrete curb, per cu. yd.	$7.653
Concrete curb, per lin. ft.	0.111
Concrete base, per cu. yd.	6.624

Detailed Post—Preparation of Foundation Course.	Cost per sq. yd.
Materials—	
1½-in. limestone, 9 cu. yds., at $2.125	$0.0413
Labor—	
Item:	
Scarifying	$0.0210
Shaping	0.0607
462.5 sq. yds., at $0.1230	$0.1230
Concrete base—	
Materials—	Cost per sq. yd.
Item:	
Portland cement, 51.75 bbls., at $1.30	$0.1455
Sand, 24.7 cu. yds., at $1.885	0.1006
Crushed pebbles, 49.8 cu. yds., at $1.8125	0.1951
Labor	0.2954
462.5 sq. yds. at	$0.7366
Concrete Curb—	
Materials—	Cost per sq. yd.
Item:	
Portland cement, 7.25 bbls., at $1.30	$0.0204
Sand, 3.6 cu. yds., at $1.885	0.0147
Crushed pebbles, 7.1 cu. yds., at $1.8125	0.0278
Forms, etc., 250 ft. B. M., at $28	0.0131
Labor	0.0416
462.5 sq. yds., at	$0.1196
Sand Cushion—	Cost per sq. yd.
Materials—	
Sand, 24.6 cu. yds., at $1.885	$0.1002
Labor	0.0376
462.5 sq. yds. at	$0.1378
Vitrified Blocks—	Cost per sq. yd.
Materials—	
Vitrified blocks, 19,466 at $29.85 per M.	$1.2563
	Cost per sq. yd.
Laying Vitrified Blocks—	
Labor—Laying and rolling, 462.5 sq. yds. at	$0.1095
Grouting and Covering—	
Materials—	Cost per sq. yd.
Item:	
Portland cement, 12.5 bbls., at $1.30	$0.0351
Sand, 9 cu. yds. at $1.885	0.0367
Labor	0.0349
462.5 sq. yds. at	$0.1067
	Cost per sq. yd.
Trimming Shoulders—	
Labor—462.5 sq. yds. at	$0.636

SECTION 5, STATION 52 + 50 TO 59 + 50. FIVE-INCH CONCRETE PAVEMENT WITH BITUMINOUS TOP.

Foundation Course.—The foundation course was originally old waterbound macadam with a telford base. This was removed to a depth of 5½ ins. below the finished grade, and where the original surface was below this grade, the surface was loosened and 1½-in. stone and screenings were spread and rolled to raise the surface to the required grade for the laying of the concrete base.

Concrete Pavement.—A concrete pavement 5 ins. thick, proportioned 1-3-6 and mixed in a No. 2 Ransome Concrete Mixer with a charging device was laid on the prepared subgrade. The surface of the concrete was finished with a template and trowels and was roughened with a stiff broom. One-quarter-inch transverse contraction joints, at right angles to the roadway, were provided at intervals of 40 ft. For this purpose ¼-in. iron plates were embedded in the concrete and removed as soon as the concrete had hardened. After the concrete was thoroughly set, the joints were filled with Pioneer Road Surface Asphalt.

Bituminous Top.—From station 52+50 to 54+58 a bituminous top of Ugite was applied. After sweeping the surface of the concrete, 1/6 gal. per square yard of Ugite "A" was applied by hand and ⅜ gal. per square yard of Ugite No. 3 was then applied at a temperature of 280° by a pressure distributor. A coating of ¾ in. dry trap rock chips followed. Another application of ¼ gal. per square yard of Ugite No. 3 was then applied at a temperature of 280° by a pressure distributor, which in turn was covered with torpedo sand and rolled with a 12-ton 3-wheel roller.

From station 54+58 to 57+17 a bituminous top of Tarvia was applied. After sweeping the surface of the concrete, ¼ gal. per square yard of Tarvia "B" was applied cold, after which ¾ gal. per square yard of Tarvia "A" was then applied at a temperature of 250° with a pressure distributor and covered with torpedo sand.

From station 57+17 to 59+50, a bituminous top of Texaco Asphalt was applied. After sweeping the surface of the concrete, ⅛ gal. per square yard of Texaco Asphalt cut back with naphtha was applied by hand, after which 6/10 gal. per square yard of Texaco Asphalt, 55 penetration, was applied by hand at a temperature of 450° and covered with clean trap rock chips passing ¾-in. screen and rolled with a 12-ton 3-wheel roller.

Cost— Item.	Cost per sq. yd.
Foundation course	$0.0816
Concrete pavement	0.7464
Shoulders	0.0594
Bituminous top, Ugite	0.1814
Bituminous top, Tarvia	0.1215
Bituminous top, Texaco	0.1632
Foundation course, concrete pavement and bituminous top, Ugite	1.0094
Foundation course, concrete pavement and bituminous top, Tarvia	0.9495
Foundation course, concrete pavement and bituminous top, Texaco	0.9912

Detailed Cost, Foundation Course—	Cost per sq. yd.
Materials—	
1½-in. limestone, 31.1 cu. yds., at $2.125	$0.0523
Labor—	
Item:	
Scarifying	$0.0071
Shaping	0.0171
Spreading stone	0.0051
1,263.1 sq. yds. at	$0.0816

Concrete Pavement— Materials— Item:	Cost per sq. yd.
Crushed pebbles, 170.3 cu. yds., at $1.8125	$0.2414
Sand, 84.3 cu. yds., at $1.885	0.1258
Portland cement, 176 bbls., at $1.30	0.1812
Lumber, 150 ft. B. M., at $28.00	0.0033
Labor	0.1917
1,263.1 sq. yds. at	$0.7464

Shoulders— Materials—	Cost per sq. yd.
Cinders, 5 tons, at $1.25	$0.0049
Labor	0.0545
1,263.1 sq. yds. at	$0.0594

Bituminous Top, Ugite, Station 52 + 50 to 54 + 58— Materials— Item:	Cost per sq. yd.
Ugite "A," 75 gals. at $0.07	$0.0134
Ugite No. 3, 250 gals., at $0.08	0.0512
¾-in. trap rock, 7 tons, at $1.90	0.0310
Torpedo sand, 6 tons, at $2.95	0.0122
Labor—	
Item:	
Hauling	$0.0266
Sweeping	0.0019
Applying bituminous materials	0.0061
Spreading chips	0.0057
391 sq. yds. at	$0.1814

Bituminous Top, Tarvia, Station 54 + 58 to 57 + 17— Materials— Item:	Cost per sq. yd.
Tarvia "B," 91 gals., at $0.07	$0.0139
Tarvia "A," 191 gals. at $0.085	0.0354
Torpedo sand, 9.5 tons, at $2.75	0.0569
Labor—	
Item:	
Hauling	$0.0085
Sweeping	0.0016
Applying bituminous materials	0.0036
Spreading chips	0.0016
459 sq. yds. at	$0.1215

Bituminous Top, Texaco, Station 57 + 17 to 59 + 50— Materials— Item:	Cost per sq. yd.
Texaco asphalt, 270 gals., at $0.085	$0.0555
Naphtha, 36 gals., at $0.18	0.0157
Trap rock chips, 6 tons, at $2.20	0.0320
Labor—	
Item.	
Hauling	$0.0072
Sweeping	0.0012
Heating bituminous materials	0.0099
Applying bituminous materials	0.0221
Spreading chips	0.0051
Rolling	0.0147
413 sq. yds. at	$0.1632

SECTION 6—STATION 59 + 50 TO 72. BITUMINOUS PAVEMENT MIXING METHOD. FILBERTINE.

Foundation Course.—The foundation course was originally old waterbound macadam with a telford base. After cleaning and spiking the old macadam, additional 1½-in. stone was added to give not less than 4 ins. of stone on the top of the telford foundation. Just enough screenings were added to form a course grainy base and the road was rolled and puddled until thoroughly cemented.

Surface Course or Paving Mixture.—The paving mixture consisted of the following mineral aggregate: ¾-in. limestone, sand and Portland cement. The mineral aggregate was combined with asphaltic cement within the following limits: Limestone, ¾-in., 55 to 65 per cent; sand, coarse to fine, and cement, 35 to 45 per cent; asphaltic cement, 70 to 80 penetration, 6 6/10 per cent. The paving mixture was prepared in a stationary plant of 600 sq. yds. capacity, erected by the contractor at the south side of the road at Station 115. The paving mixture was laid at a temperature of 250 to 350° and rolled to a finished depth of 2 ins. with a 6-ton tandem roller.

Cost—	Cost per sq. yd
Preparation of foundation course	$0.4569
Wearing surface	0.7584
Trimming shoulders	0.0428
	$1.2581

Detailed Cost, Preparation of Foundation Course— Materials— Item:	Cost per sq. yd.
1½-in. trap rock, 236 cu. yds., at $2.43	$0.2532
Trap rock screenings, 49.7 cu. yds., at $2.363	0.0514
Labor—	
Item:	
Scarifying	$0.0410
Spreading trap rock	0.0504
Rolling and watering	0.0609
2,265 sq. yds. at	$0.4569

Wearing Surface— Materials— Item:	Cost per sq. yd.
¾-in. limestone, 136.4 cu. yds., at $2.145	$0.1292
Asphalt sand, 54.2 cu. yds., at $1.74	0.0416
Coarse sand, 53.7 cu. yds., at $1.885	0.0447
Asphaltic cement, 41,110 lbs., at $0.012917	0.2344
Portland cement, 35 bbls., at $1.30	0.0201
Forms, etc., 500 ft. B. M., at $28	0.0062
Coal, 24 tons, at $4	0.0424
Wood, 1⅓ cords, at $7.50	0.0044
Labor—	
Item:	
Mixing	$0.0797
Applying bituminous materials	0.0872
Rolling	0.0186
Hauling bituminons materials	0.0499
2,265 sq. yds. at	$0.7584

Trimming Shoulders— Materials—	Cost per sq. yd.
Cinders, 5 tons, at $1.25	$0.0027
Labor	0.0401
2,265 sq. yds. at	$0.0428

SECTION 7—STATION 72 TO 84 + 50. BITUMINOUS MACADAM MIXING METHOD.

Foundation Course.—The foundation course was originally old waterbound macadam with a telford base. After cleaning and spiking the old macadam, additional 1½-in. stone was added to give not less than 4 ins. of stone on the top of the telford foundation. Just enough screenings were added to form a coarse grainy base and the road was rolled and puddled until thoroughly cemented.

Surface Course or Paving Mixture.—The paving mixture consisted of the following mineral aggregate: Trap rock, run of crusher and passing a 1¼-in. screen, 2 parts; sand, 1 part, mineral dust, 5 per cent of the above, asphaltic cement, 5.2 per cent.

This mineral aggregate was mixed with the asphaltic cement in a stationary asphalt plant erected by the contractor at the south side of the road at Station 115. The paving mixture was laid at a temperature of 250 to 350° and rolled to a finished depth of 2 ins. with a 6-ton tandem roller. After the rolling was completed that portion not covered by the seal coat was swept and covered with dry sand.

Seal Coat.—From Station 72 to 78 a seal coat of ¼ gal. per square yard of asphaltic cement was applied at a temperature of 350° and covered with clean trap rock chips, passing ¾-in. screen and rolled.

Cost— Item:	Cost per sq. yd.
Foundation course	$0.5067
Wearing surface	0.7934
Sand covering	0.0073
Trimming shoulders	0.0200
	$1.3274
Seal coat	0.1017
Cost, including seal coat	$1.4291

Detailed Cost, Preparation of Foundation Course— Materials— Item:	Cost per sq. yd.
1½-in. trap rock, 243.4 cu. yds., at $2.43	$0.2620
Trap rock screenings, 49.5 cu. yds., at $2.295	0.0503
Labor—	
Item:	
Scarifying	$0.0375
Spreading stone	0.0794
Rolling and watering	0.0775
2,257.71 sq. yds. at	$0.5067

Wearing Surface— Materials— Item:	Cost per sq. yd.
Trap rock, 1¼-inch and smaller, 162.5 cu. yds., at $2.43	$0.1749
Coarse sand, 19.9 cu. yds., at $1.885	0.0166
Asphalt sand, 37.1 cu. yds., at $1.74	0.0286
Asphaltic cement, 31,000 lbs., at $0.012867	0.1774
Portland cement, 48 bbls., at $1.30	0.0276
Lumber, 500 ft. B. M., at $28	0.0062
Coal, 32 tons, at $4	0.0567
Wood, 1⅓ cords, at $7.50	0.0044
Labor—	
Item:	
Mixing	$0.1129
Hauling bituminous materials	0.0635
Applying bituminous materials	0.1000
Rolling bituminous materials	0.0246
2,257.71 sq. yds., at	$0.7934

Seal Coat— Materials— Item:	Cost per sq. yd.
Trap rock chips, 13.3 cu. yds., at $3.44425	$0.0425
Asphaltic cement, 2,802 lbs., at $0.012867	0.0336
Labor—	
Item:	
Sweeping	$0.0006
Applying bituminous materials	0.0136
Chipping	0.0088
Rolling	0.0026
1,077.06 sq. yds., at	$0.1017

Sand Covering— Materials— Item:	Cost per sq. yd.
Sand, 4.1 cu. yds., at $1.885	$0.0065
Labor	0.0008
1,180.65 sq. yds., at	$0.0073

Trimming Shoulders— Materials— Item:	Cost per sq. yd.
Cinders, 5 tons, at $1.25	$0.0025
Labor	0.0175
2,257.71 sq. yds., at	$0.0200

SECTION 8—STATION 84 + 50 TO 87 + 50. VITRIFIED BLOCK PAVEMENT ON 4-IN. CONCRETE BASE.

Foundation Course.—The foundation course was originally old waterbound macadam with a telford base. The surface was graded to within 9½ ins. of the finished grade, and additional material was added where required. It was then thoroughly rolled.

Concrete Base and Vitrified Blocks.—A 4-in. concrete base with concrete curbs, proportioned 1-3-6, was laid upon the prepared subgrade. The concrete was mixed in a Ransome No. 2 Concrete Mixer, equipped with a charging device.

The sand cushion was spread and rolled with an iron hand roller and shaped with a template. After laying, the vitrified blocks were rolled with a 6-ton tandem roller, and grouted to within ½ in. of the surface with cement grout, proportioned 1-1, and covered with sand.

Cost— Item	Cost per sq. yd.
Preparation of foundation course	$0.1957
Concrete base	0.6866
Concrete curb	0.1142
Sand cushion	0.1176
Vitrified blocks	1.2559
Laying vitrified blocks	0.1021
Grouting and covering	0.0906
Trimming shoulders	0.0402
	$2.6032
Concrete base, per cubic yard	$6.179
Concrete curb, per cubic yard	6.943
Concrete curb, per linear foot	0.102

Detailed Preparation of Foundation Course— Materials— Item	Cost per sq. yd.
1½-inch limestone, 27.6 cu. yds., at $2	$0.1031

Labor—	
Item:	
Scarifying	$0.0137
Shaping	0.0528
Spreading 1½-in. limestone	0.0084
Rolling and watering	0.0177
535.5 sq. yds. at	$0.1957

Concrete Base—

Materials—	Cost per sq. yd.
Item:	
Crushed pebbles, 57.7 cu. yds., at $1.8125	$0.1953
Sand, 28.4 cu. yds., at $1.885	0.1000
Portland cement, 60 bbls., at $1.30	0.1456
Labor	0.2457
535.5 sq. yds. at	$0.6866

Concrete Curb—

Materials—	Cost per sq. yd.
Item:	
Crushed pebbles, 8.5 cu. yds., at $1.8125	$0.0287
Sand, 4.2 cu. yds., at $1.885	0.0147
Portland cement, 8.75 bbls., at $1.30	0.0213
Forms, etc., 250 ft. B. M., at $28	0.0131
Labor	0.0364
535.5 sq. yds., at	$0.1142

Sand Cushion—

Materials—	Cost per sq. yd.
Item:	
Sand, 29.6 cu. yds., at $1.885	$0.1042
Labor	0.0134
535.5 sq. yds., at	$0.1176

Vitrified Blocks—

Materials—	Cost per sq. yd.
Item:	
Vitrified blocks, 22,530 at $29.35 per M.	$1.2559
535.5 sq. yds., at	$1.2559

Laying Vitrified Blocks—

Item:	Cost per sq. yd.
Labor, 535.5 sq. yds., at	$0.1024

Grouting and Covering—

Materials—	Cost per sq. yd.
Item:	
Portland cement, 15 bbls., at $1.30	$0.0364
Sand, 9.5 cu. yds., at $1.885	0.0334
Labor	0.0208
535.5 sq. yds., at	$0.0906

Trimming Shoulders—

Materials—	Cost per sq. yd.
Item:	
Cinders, 5 tons, at $1.25	$0.0117
Labor	0.0285
535.5 sq. yds., at	$0.0402

SECTION 9—STATION 87 + 50 TO 94. FIVE-IN. CONCRETE PAVEMENT WITH BITUMINOUS TOP.

Foundation Course.—The foundation course was originally old waterbound macadam with a telford base. This was removed to a depth of 5½ ins. below the finished grade, and where the original surface was below this grade the surface was spiked and 1½-in. stone and screenings were spread and rolled to raise the surface to the proper grade.

Concrete Pavement.—A concrete base, 5 ins. thick, was laid upon the subgrade. This concrete was proportioned 1-3-6 and was mixed by a No. 2 Ransome Concrete Mixer equipped with a charging device. The surface of the concrete was finished with a template and trowels and was roughened with a stiff broom.

Transverse contraction joints, at right angles to the line of roadway, were provided at intervals of 10 ft. For this purpose two pieces of tar felt paper were embedded in the concrete at each joint and trimmed to the grade of the concrete surface when it was finished.

Bituminous Top. From station 87 + 50 to 92, a bituminous top was laid. From station 87 + 50 to 90, 4/10 gal. per square yard of Dolarway Bitumen was applied at a temperature of 270° and covered with torpedo sand.

From station 90 to 90 + 70, ⅛ gal. per square yard of Bicomac was spread on the surface of the concrete pavement. A mixture of trap rock chips, trap rock dust and Bicomac was then applied and rolled with a hand roller. One-half of this surface was covered with trap rock dust and rolled lightly. The other half was covered with trap rock chips and rolled lightly.

From station 90 + 70 to 91 + 35, 1/10 gal per square yard of asphalt cut back with naphtha was spread upon the surface of the concrete pavement and the naphtha was burned out. Trap rock chips were then spread and rolled into this asphalt coating with a hand roller.

From station 91 + 35 to 92, ⅛ gal per square yard of Bicomac was spread upon the surface of the concrete pavement, after which 4/10 gal per square yard of asphalt was applied at a temperature of 400° and covered with clean trap rock chips passing ¾-in. screen.

Cost—	Cost per sq. yd.
Item:	
Preparation of foundation course	$0.1680
Concrete pavement	0.8792
Trimming shoulders	0.0391
	$1.0863
Bituminous top Dolarway	0.1689
Bituminous top Bicomac mixing method	0.1203
Bituminous top cut back asphalt fired	0.1057
Bituminous asphalt and Bicomac	0.1004

Detailed Cost Preparation of Foundation Course—

Materials:	Cost per sq. yd.
1½-in. limestone, 524 cu. yds., at $2.00	$0.0845
Labor—	
Item:	
Scarifying	$0.0577
Shaping	0.0012
Spreading stone	0.0202
Rolling and watering	0.0044
1,240.7 sq. yds. at	$0.1680

Concrete Base—

Materials—	Cost per sq. yd.
Item:	
Crushed pebbles, 167.6 cu. yds., at $1.8125	$0.2448
Gravel, 82.8 cu. yds., at $1.885	0.1258
Portland cement, 173 bbls. at $1.30	0.1813
Forms, etc., 300 ft., B. M., at $28.00	0.0068
Labor	0.3205
1,240.7 sq. yds. at	$0.8792

Trimming Shoulders—

Materials:	Cost per sq. yd.
Cinders, 15 tons at $1.25	$0.0151
Labor	0.0240
1,240.7 sq. yds. at	$0.0391

Bituminous Top Station 87 + 50 to 90 Dolarway.

Materials—	Cost per sq. yd.
Item:	
Dolarway bitumen, 200 gallons, at $0.095	$0.0417
Torpedo sand, 7 tons, at $2.75	0.0422
Labor:	
Hauling	$0.0482
Heating bituminous materials	0.0055
Applying bituminous materials	0.0184
Chipping bituminous materials	0.0129
456 sq. yds. at	$0.1689

Bituminous Top Station 90 to 90 + 90 Bicomac, Mixing Method—

Materials—	Cost per sq. yd.
Item:	
Bicomac, 72 gallons, at $0.100	$0.0581
½-inch trap rock, and Trap rock chips, 2½ tons, at $2.20	0.0444
Labor	0.0274
124 sq. yds. at	$0.1299

Bituminous Top Station 90 + 70 to 91 + 35 Cut Back Asphalt. Fired—

Materials—	Cost per sq. yd.
Item:	
Bicomac, 7 gallons, at $0.10	$0.0061
Cut back asphalt, 58 gallons, at $0.11	0.0555
Trap rock chips, 1½ tons, at $2.20	0.0287
Labor	0.0274
115 sq. yds., at	$0.1177

Bituminous Top Station 91 + 35 to 92 Asphalt and Bicomac—

Materials—	Cost per sq. yd.
Item:	
Bicomac, 46 gallons, at $0.10	$0.0241
Asphalt, 60 gallons, at $0.115	0.0362
Trap rock chips, 2 tons, at $2.20	0.0230
Labor	0.0274
191 sq. yds., at	$0.1107

SECTION 10—STATION 94 TO 97. VITRIFIED BLOCK PAVEMENT ON 4-IN. CONCRETE BASE.

Foundation Course.—The foundation course was originally old waterbound macadam with a telford base. The surface was graded to within 9½ ins. of the finished grade and additional material was added where required. It was then thoroughly rolled.

Concrete Base and Vitrified Blocks.—A 4-in. concrete base with concrete curbs, proportioned 1-3-6, was laid upon the prepared subgrade. The concrete was mixed in a Ransome No. 2 concrete mixer equipped with a charging device.

The sand cushion was spread and rolled with an iron hand roller, and shaped with a template. After laying, the vitrified blocks were rolled with a 6-ton tandem roller and grouted to within ½-in. of the surface with cement grout, proportioned 1-1, and covered with sand.

Cost—	Cost per sq. yd.
Item:	
Preparation of foundation course	$0.1125
Concrete base	0.6898
Concrete curb	0.1151
Sand cushion	0.1176
Vitrified blocks	1.2560
Laying vitrified blocks	0.0777
Grouting and covering	0.1047
Trimming shoulders	0.0798
	$2.5532
Concrete base, per cubic yard	$6.208
Concrete curb, per lineal foot	0.1025
Concrete curb, per cubic yard	7.001

Detailed Cost Preparation of Foundation Course—

Materials:	Cost per sq. yd.
1½-in. limestone, 10.4 cu. yds., at $2.00	$0.0389
Labor—	
Item:	
Scarifying	$0.0296
Shaping	0.0279
Spreading 1½-in. limestone	0.0066
Rolling and watering	0.0095
534.3 sq. yds. at	$0.1125

Concrete Base—

Materials—	Cost per sq. yd.
Item:	
Crushed pebbles, 57.5 cu. yds., at $1.8125	$0.1951
Sand, 28.4 cu. yds., at $1.885	0.1002
Portland cement, 59 bbls., at $1.30	0.1435
Labor	0.2510
534.3 sq. yds. at	$0.6898

Concrete Curb—

Materials—	Cost per sq. yd.
Item:	
Crushed pebbles, 8.5 cu. yds., at $1.8125	$0.0288
Sand, 4.2 cu. yds., at $1.885	0.0148
Portland cement, 8.75 bbls., at $1.30	0.0213
Forms, etc., 250 ft. B. M., at $28.00	0.0131
Labor	0.0371
534.3 sq. yds. at	$0.1151

Sand Cushion—

Materials:	Cost per sq. yd.
Sand, 29.6 cu. yds., at $1.885	$0.1044
Labor	0.0132
534.3 sq. yds. at	$0.1176

Vitrified Blocks—

Material:	Cost per sq. yd.
Vitrified blocks, 22482 at $29.85 per M.	$1.2560
534.3 sq. yds. at	1.2560

Laying Vitrified Blocks—

Labor—laying and rolling 534.3 sq. yds. at	$0.0777

Grouting and Covering—

Materials—	
Item:	
Portland cement, 15 bbls., at $1.30	$0.0365
Sand, 9.4 cu. yds., at $1.885	0.0332
Labor	0.0350
534.3 sq. yds. at	$0.1047

Trimming Shoulders—

Materials:	
Cinders, 5 tons, at $1.25	$0.0117
Labor	0.0681
534.3 sq. yds. at	$0.0798

SECTION 11—SECTION 97 TO 103. BITUMINOUS PAVEMENT MIXING METHOD.

Foundation Course.—The foundation course was originally old waterbound macadam with a telford base. After cleaning and spiking the old macadam, additional 1½-in. stone was added to give not less than 4 ins. on the top of the telford foundation. Just enough screenings were added to form a coarse grainy base and the road was rolled and puddled until thoroughly cemented. A 12-ton 3-wheel roller was used.

Surface Course or Paving Mixture.—The paving mixture consisted of the following mineral aggregate: Trap rock, run of crusher, passing a 1-in. screen and retained on a ¼-in. screen. The mineral aggregate was mixed with 4.8 per cent of asphaltic cement in a stationary asphalt plant erected by the contractor at the south side of the road at Station 115. The paving mixture was laid at a temperature of 250 to 350° and rolled to a finished depth of 2 ins. with a 6-ton tandem roller.

Seal Coat.—A Seal Coat of ¼ gal. per square yard of asphaltic cement was applied at a temperature of 350°, and covered with clean trap rock chips passing a ⅜-in. screen and rolled.

Cost—	Cost per sq. yd.
Item:	
Foundation course	$0.4934
Wearing surface	0.6460
Seal coat	0.1050
Trimming shoulders	0.0392
	$1.2836

Detailed Cost Foundation Course—

Materials—	Cost per sq. yd.
Item:	
1½-in. trap rock, 146.3 cu. yds., at $2.43	$0.3186
Trap rock screenings, 24.5 cu. yds., at $2.295	0.0504
Labor—	
Item:	
Scarifying	$0.0396
Shaping	0.0154

Spreading stone..	0.0224
Rolling and watering.	0.0470
1115.9 sq. yds. at...	$0.4934
Wearing Surface—	
Materials—	Cost per sq. yd.
Item:	
¾-in. trap rock, 92 cu. yds., at $2.43.	$0.2003
Asphaltic cement, 12040 lbs., at $0.012867	0.1394
Coal, 16 tons, at $4.00	0.0574
Wood, ⅔ cords, at $7.50	0.0045
Forms, etc., 200 ft. B. M., at $28.00	0.0050
Labor—	
Item:	
Mixing	0.0776
Hauling bituminous materials	0.0542
Applying bituminous materials	0.0915
Rolling	0.0161
1115.9 sq. yds. at	$0.6460
Seal Coat—	
Materials—	Cost per sq. yd.
Item:	
Trap rock chips, 13.3 cu. yds., at $3.4125	$0.0410
Asphaltic cement, *3340 lbs., at $0.012867	0.0387
Labor—	
Item:	
Sweeping	$0.0024
Heating bituminous materials	0.0059
Applying bituminous materials	0.0066
Chipping	0.0053
Rolling	0.0051
1115.9 sq. yds. at	$0.1050
Trimming Shoulders—	Cost per sq. yd.
Materials:	
Cinders, 10 tons, at $1.25.	$0.0112
Labor	0.0280
1115.9 sq. yds. at	$0.0392

SECTION 12—STATION 103 TO 109. BITUMINOUS PAVEMENT MIXING METHOD.

Foundation Course.—The foundation course was originally old waterbound macadam with a telford base. After cleaning and spiking the old macadam, additional 1½-in. stone was added to give not less than 4 ins. of stone on the top of the telford foundation. Just enough screenings were added to form a coarse grainy base and the road was rolled and puddled until thoroughly cemented. A 12-ton 3-wheel roller was used.

Surface Course or Paving Mixture.—The paving mixture consisted of the following mineral aggregate: Bank gravel, with no stone over 1 in. in its largest dimensions, and sand.

The mineral aggregate was mixed with 8.1 per cent of asphaltic cement in a stationary asphalt plant erected by the contractor at the roadside at Station 115. The paving mixture was laid at a temperature of 250 to 350° and rolled to a finished depth of 2 ins. with a 6-ton tandem roller.

From Station 103 to 106, a seal coat of ¼ gal. per square yard of asphaltic cement was applied at a temperature of 350° and covered with clean trap rock chips passing a ⅜-in screen and rolled.

From Station 106 to 109 the surface of the road was covered with sand.

Cost—	Cost per sq. yd.
Item:	
Foundation course	$0.4889
Wearing surface	0.8542
Sand covering	0.0086
Trimming shoulders	0.1288
	$1.4805
Seal coat	0.1278
Cost, including seal coat	$1.6083
Detailed Cost Foundation Course—	
Materials—	Cost per sq. yd.
Item:	
1½-in. trap rock, 141 cu. yds., at $2.43	$0.3096
Trap rock screenings, 21.3 cu. yds., at $2.295	0.0504
Labor—	
Item:	
Scarifying	$0.009[illegible]
Shaping	0.015[illegible]
Spreading stone	[illegible].0517
Rolling and watering	[illegible].0525
1106.67 sq. yds. at	$0.4889
Wearing Surface	
Materials—	
Item:	
Bank gravel, 97.2 cu. yds., at $2.[illegible]	$0.2141
Sand, 25.4 cu. yds., at $1.71	0.0399
Asphaltic cement, 25310 lbs., at $0.012917	0.2954
Coal, 16 tons, at $4.00	0.0578
Wood, ⅔ cords, at $7.50	0.0045
Forms, etc., 200 ft. B. M., at $28.00	0.0051
Labor—	
Item:	
Mixing	$0.0660
Hauling bituminous materials	0.0518
Applying bituminous materials.	0.1005
Rolling	0.0191
1106.67 sq. yds. at	$0.8542
Seal Coat—	
Materials—	
Item:	
Trap rock chips, 6.7 cu. yds., at $3.4425	$0.0415
Asphaltic cement, 2858 lbs., at $0.012917	0.0665
Labor—	
Item:	
Heating bituminous materials	$0.0045
Applying	0.0049
Chipping	0.0065
Rolling	0.0039
553.4 sq. yds. at	$0.1278
Sand Covering	
Materials:	
Sand, 1.9 cu. yds., at $1.885	$0.0065
Labor	0.0021
553.4 sq. yds. at	$0.0086
Trimming Shoulders—	
Materials:	
Cinders, 55 tons, at $1.25	$0.0621
Labor	0.0667
1106.67 sq. yds. at	$0.1288

SECTION 13—STATION 109 TO 112 + 50. VITRIFIED BLOCK PAVEMENT ON 4-IN. CONCRETE BASE.

Foundation Course.—The foundation course was originally old waterbound macadam with a telford base. The surface was graded to within 9½ ins. of the finished grade and additional material was added where required. It was then thoroughly rolled.

Concrete Base and Vitrified Blocks.—A 4-in. concrete base with concrete curbs, proportioned 1-3-6, was laid upon the prepared subgrade. The concrete was mixed in a Ransome No. 2 concrete mixer, equipped with a charging device.

The sand cushion was spread and rolled with an iron hand roller, and shaped with a template. After laying, the vitrified blocks were rolled with a 6-ton tandem roller and grouted to within ½ in. of the surface with cement grout proportioned 1-1 and covered with sand.

Cost—	Cost per sq. yd.
Item:	
Preparation of foundation course	$0.3044
Concrete base	0.7446
Concrete curb	0.1235
Sand cushion	0.1309
Vitrified blocks	1.2557
Laying vitrified blocks	0.1288
Grouting and covering	0.0883
Trimming shoulders	0.0681
	$2.8443
Concrete base, per cubic yard	$6.702
Concrete curb, per cubic yard	7.496
Concrete curb, per linear foot	0.1096
Detailed Cost Preparation of Foundation Course—	Cost per sq. yd.
Materials:	
1½-in. limestone, 45.6 cu. yds., at $2.00	$0.1468
Labor	
Item:	
Scarifying	$0.0672
Shaping	0.0566
Spreading 1½-in limestone	0.0137
Rolling and watering	0.0201
621.4 sq. yds. at	$0.3044
Concrete Base—	
Materials—	
Item:	
Crushed pebbles, 67 cu. yds., at $1.8125	$0.1951
Sand, 33.1 cu. yds., at $1.885.	0.1004
Portland cement, 69.75 bbls., at $1.30.	0.1459
Labor	0.3029
621.4 sq. yds. at	$0.7446
Concrete Curb—	
Materials	
Item:	
Crushed pebbles, 10.1 cu. yds., at $1.8125	$0.[illegible]
Sand, 1.5 cu. yds., at $1.885	[illegible]
Portland cement, 10.[illegible] bbls., at $[illegible]	[illegible]
Forms, etc., 200 ft. B. M., at $28.00	[illegible]
Labor	0.0[illegible]
621.4 sq. yds. at	$0.12[illegible]
Sand Cushion—	
Materials	
Sand, [illegible] cu. yds., at $1.8[illegible]	$[illegible]
Labor	[illegible]
621.4 sq. yds. at	$[illegible]
Vitrified Blocks—	
Materials	
Vitrified blocks, 26,110 at $[illegible] per M.	$[illegible]
621.4 sq. yds. at	1.2557
Laying Vitrified Blocks—	
Labor	
Laying and rolling 621.4 sq. yds. at	$0.1288
Grouting and Covering—	
Materials—	
Item:	
Portland cement, 17.5 bbls., at $1.30.	$0.0366
Sand, 10.9 cu. yds., at $1.885	0.0331
Labor	0.0186
621.4 sq. yds. at	$0.0883
Trimming Shoulders—	
Materials:	
Cinders, 5 tons, at $1.25	$0.0101
Labor	0.0580
621.4 sq. yds. at	$0.0681

SECTION 14—STATION 112 + 50 TO 118. FIVE-INCH HASSAM CONCRETE PAVEMENT WITH BITUMINOUS TOP.

Foundation Course.—The foundation course was originally old waterbound macadam with a telford base. This was removed to a depth of 5½ ins. below the finished grade and where the original surface was lower than this grade, the surface was loosened and 1½-in. stone and screenings were spread and rolled to raise the surface to the required grade for the laying of the concrete pavement.

Concrete Pavement.—Stone was spread upon the foundation coarse and rolled with a 6-ton tandem roller to a finished depth of 5 ins. From Station 112 + 50 to 114 + 85, 2½-in. trap rock was used.

From Station 114 + 85 to 116, 1½-in. trap rock was used.

From Station 116 to 118, crushed pebbles in size from 1 to 2 ins. were used.

Cement grout, proportioned 1-2, was mixed in a Hassam Grout Mixer and poured on the stone until the grout flushed to the surface. As soon as the grout had settled the road was rolled with a 6-ton tandem roller.

Bituminous Top.—A bituminous top was laid upon the concrete surface. One-fourth gal. per square yard of Ugite No. 3 was applied at a temperature of 250°, by a pressure distributor which was covered with clean trap rock chips passing ½-in. screen and rolled with a 6-ton tandem roller, after which ¼ gal. per square yard of an asphaltic cement was then applied at a temperature of 425° by a pressure distributor and covered with clean trap rock chips passing ½-in. screen and rolled.

Cost—	Cost per sq. yd.
Item:	
Preparation of foundation course	$0.2255
Concrete pavement	0.8919
Bituminous top	0.1331
	$1.2505
Detailed Cost Preparation of Foundation Course—	Cost per sq. yd.
Materials:	
1½-in. limestone, 48 cu. yds., at $2.00	$0.0977
Labor—	
Item:	
Shaping	0.0572
Spreading stone	0.0362
Rolling and watering	0.0344
983.1 sq. yds. at	$0.2255
Grouted Concrete Pavement—	
Materials—	Cost per sq. yd.
Item:	
1½-in. trap rock, 28.1 cu. yds., at $2.43	$0.0702
2½-in. trap rock, 58.9 cu. yds., at $2.43	0.1456
Crushed pebbles, 44.9 cu. yds., at $1.8125	0.0911
Sand, 46.7 cu. yds., at $[illegible].74	0.0827
Portland cement, 175 lbs., at $1.30	0.2314
Forms, etc., 275 ft. B. M., at $28.00	0.0078
Labor—	
Item:	
Concrete pavement	0.0991
Grouting	[illegible].1640
983.1 sq. yds. at	$0.8919
Bituminous Top—	
Materials—	Cost per sq. yd.
Item:	
Ugite No. 3, 246 gals., at $0.08.	$0.0200
Asphaltic cement, 246 gals., at $0.1095	[illegible].0274
Trap Rock chips, 20 tons, at $2.20	0.0447
Labor—	
Item:	
Sweeping	0.0059
Heating bituminous materials	0.011[illegible]
Applying bituminous materials	0.0063
Spreading chips	0.0097
Rolling	0.0076
983.1 sq. yds. at	$0.1331

SECTION 15—STATION 118 TO 121. VITRIFIED BLOCK PAVEMENT ON 4-IN. CONCRETE BASE.

Foundation Course.—The foundation course was originally old waterbound macadam with a telford base. The surface was graded to within 9½ ins. of the finished grade and additional material was added where required. It was then thoroughly rolled.

Concrete Base and Vitrified Blocks.—A 4-in. concrete base with concrete curbs, proportioned 1-3-6, was laid upon the prepared subgrade. The concrete was mixed in a Ransome No. 2 concrete mixer, equipped with a charging device.

The sand cushion was spread, rolled with an iron hand roller, and shaped with a template. After laying, the vitrified blocks were rolled with a 6-ton tandem roller, grouted to within ½ in. of the surface with cement grout, proportioned 1-1, and covered with sand.

Cost	Cost per sq. yd.
Item:	
Preparation of foundation course	$0.1618
Concrete base	0.6775
Concrete curb	0.1116
Sand cushion	0.1270
Vitrified block	1.2560
Laying vitrified blocks	0.0446
Grouting and covering	0.0975
Trimming shoulders	0.1641
	$2.6401
Concrete base, per cubic yard	$6.098
Concrete curb, per lineal foot	0.0992
Concrete curb, per cubic yard	6.786

Detailed Cost Preparation of Foundation Course—	Cost per sq. yd.
Materials:	
1½-in. limestone, 7 cu. yds., at $2.00	$0.0262
Labor—	
Item:	
Scarifying	$0.0903
Shaping	0.0367
Rolling and watering	0.0086
534.3 sq. yds. at	$0.1618

Concrete Base—	Cost per sq. yd.
Materials—	
Item:	
Crushed pebbles, 57.6 cu. yds., at $1.8125	$0.1954
Sand, 28.4 cu. yds., at $1.885	0.1002
Portland cement, 60 bbls., at $1.30	0.1460
Labor	0.2359
534.3 sq. yds. at	$0.6775

Concrete Curb—	
Materials—	
Item:	
Crushed pebbles, 8.1 cu. yds., at $1.8125	$0.0275
Sand, 4.2 cu. yds., at $1.885	0.0148
Portland cement, 8.75 bbls., at $1.30	0.0213
Forms, etc., 250 ft. B M., at $28.00	0.0131
Labor	0.0349
534.3 sq. yds. at	$0.1116

Sand Cushion—	
Materials:	
Sand, 29.6 cu. yds., at $1.885	$0.1044
Labor	0.0226
534.3 sq. yds. at	$0.1270

Vitrified Blocks—	
Materials:	
Vitrified blocks, 22,482, at $29.85 per M, 534.3 sq. yds. at	$1.2560

Laying Vitrified Blocks—	
Labor:	
Laying and rolling 534.3 sq. yds. at	$0.0446

Grouting and Covering	Cost per sq. yd.
Materials	
Item:	
Portland cement, 15 bbls., at $1.30	$0.0365
Sand, 9.4 cu. yds., at $1.885	0.0332
Labor	0.0278
534.3 sq. yds. at	$0.0975

Trimming Shoulders—	
Materials:	
Cinders, 5.5 tons, at $1.25	$0.0129
Labor	0.1512
534.3 sq. yds. at	$0.1641

SECTION 16—STATION 121 TO 127. BITUMINOUS PAVEMENT, PENETRATION METHOD, UGITE BINDER.

Foundation Course.—The foundation course was originally old waterbound macadam with a telford base. After cleaning and spiking the old macadam, 1½-in. trap rock was spread and rolled to bring the surface to a grade 3 ins. below the finished grade. Screenings were then added and the road rolled and puddled until thoroughly cemented. A 12-ton 3-wheel roller was used.

Wearing Surface.—Three ins. of 1½-in. trap rock was spread upon the foundation course and rolled. One and six-tenths gals. per square yard of Ugite Binder No. 3, heated in tar kettles to a temperature of 250°, was then applied by hand and covered with clean ¾-in. trap rock chips and rolled with a 12-ton 3-wheel roller.

Seal Coat.—A seal coat of 6/10 of a gal. per square yard of Ugite No. 3 was applied by hand at a temperature of 250° and covered with clean ½-in. trap rock chips and rolled.

Cost—	Cost per sq. yd.
Preparation of foundation course	$0.1400
Wearing surface	0.5042
Seal coat	0.1762
Trimming shoulders	0.0662
	$0.8866

Detailed Cost Preparation of Foundation Course—	Cost per sq. yd.
Materials—	
Item:	
1½-in. trap rock, 35.3 cu. yds., at $2.6325	$0.0847
Trap rock screenings, 54.3 cu. yds., at $2.6325	0.0130
Labor—	
Item:	
Scarifying	$0.0093
Shaping	0.0060
Spreading 1½-in. trap rock	0.0184
Rolling and watering	0.0086
1097 sq. yds. at	$0.1400

Wearing Surface—	Cost per sq. yd.
Materials—	
Item:	
1½-in. trap rock, 91.4 cu. yds., at $2.6325	$0.2193
¾-in. trap rock, 20.3 cu. yds., at $2.6325	0.0487
Ugite binder, 1750 gals., at $0.076	0.1212
Labor—	
Item:	
Spreading 1½-in. trap rock	$0.0320
Rolling 1½-in. trap rock	0.0204
Heating Ugite binder	0.0237
Applying Ugite binder	0.0124
Spreading ¾-in. chips	0.0087
Rolling ¾-in. chips	0.0178
1097 sq. yds. at	$0.5042

Seal Coat—	Cost per sq. yd.
Materials—	
Item:	
½-in. trap rock, 25.2 cu. yds., at $2.6325	$0.0605
Ugite binder, 650 gals. at	0.0450
Labor—	
Item:	
Heating Ugite binder	$0.0081
Applying Ugite binder	0.0055
Spreading ½-in. chips	0.0098
Rolling ½-in. chips	0.0473
1097 sq. yds. at	$0.1762

Trimming Shoulders—	
Labor: 1097 sq. yds. at	$0.0662

SECTION 17—STATION 127 TO 131 + 60. BITUMINOUS PAVEMENT, PENETRATION METHOD, BYERLITE ASPHALT.

Foundation Course.—The foundation course was originally old waterbound macadam with a telford base. After cleaning and spiking the old macadam, 1½-in. trap rock was spread and rolled to bring the surface to a grade 3 ins. below the finished grade. Screenings were then added and the road rolled and puddled until thoroughly cemented. A 12-ton 3-wheel roller was used.

Wearing Surface.—Three ins. of 1½-in. trap rock was spread upon the foundation course and rolled. One and seven-tenths gals. per square yard of Byerlite Asphalt, heated in tar kettles to a temperature of 400°, was then applied by hand and covered with clean ¾-in. trap rock chips and rolled with a 12-ton 3-wheel roller.

Seal Coat.—A seal coat of ½ gal. per square yard of Byerlite Asphalt was applied by hand at a temperature of 400° and covered with clean ½-in. trap rock chips and rolled.

Cost—	Cost per sq. yd.
Item:	
Preparation of foundation course	$0.1523
Wearing surface	0.6827
Seal coat	0.2183
Trimming shoulders	0.0066
	$1.0599

Detailed Cost Preparation of Foundation Course—	Cost per sq. yd.
Materials—	
Item:	
Trap rock, 16.1 cu. yds., at $2.6325	$0.0656
Trap rock screenings, 2.4 cu. yds., at $2.6325	0.0098
Labor:	
Item:	
Scarifying	$0.0023
Shaping	0.0239
Spreading trap rock	0.0246
Rolling and watering	0.0261
	$0.1523

Wearing Surface—	Cost-per sq. yd.
Materials—	
Item:	
1½-in. trap rock, 53.9 cu. yds., at $2.6325	$0.2195
¾-in. trap rock, 12 cu. yds., at $2.6325	0.0489
Byerlite asphalt, 1077 gals., at $0.1043	0.1738
Labor—	
Item:	
Spreading 1½-in. trap rock	$0.0606
Rolling 1½-in. trap rock	0.0742
Heating Byerlite asphalt	0.0453
Applying Byerlite asphalt	0.0207
Spreading ¾-in. chips	0.0150
Rolling ¾-in. chips	0.0247
646.3 sq. yds. at	$0.6827

Seal Coat—	Cost per sq. ft.
Materials—	
Item:	
½-in. trap rock, 14.9 cu. yds., at $2.6325	$0.0607
Byerlite asphalt, 323 gals., at $0.1043	0.0522
Labor—	
Item:	
Heating Byerlite asphalt	$0.0132
Applying Byerlite asphalt	0.0084
Spreading ½-in. chips	0.0151
Rolling ½-in. chips	0.0687
646.3 sq. yds. at	$0.2183

Trimming Shoulders	Cost per per yd.
Labor 646.3 sq. yds. at	$0.0066

SECTION 18—STATION 130 + 60 TO 133. VITRIFIED BLOCK PAVEMENT ON 4-IN. CONCRETE BASE.

Foundation Course.—The foundation course was originally old waterbound macadam with a telford base. The surface was graded to within 9½ ins. of the finished grade and additional material was added where required. It was then thoroughly rolled.

Concrete Base and Vitrified Blocks.—A 4-in. concrete base with concrete curbs, proportioned 1-3-6, was laid upon the prepared subgrade. The concrete was mixed by hand labor.

The sand cushion was spread, rolled and shaped with a template after laying, the vitrified blocks were rolled and grouted to within ½ in. of the surface with cement grout proportioned 1-1 and covered with sand.

Cost—	Cost per sq. ft.
Item:	
Foundation course	$0.1912
Concrete base	0.8758
Concrete curb	0.1702
Sand cushion	0.1425
Vitrified blocks	1.1557
Laying vitrified blocks	0.1787
Grouting and covering	0.1137
Trimming shoulders	0.0392
	$2.8670
Concrete base, per cu. yd	$7.8801
Concrete curb, per linear ft	0.1524
Concrete curb, per cubic yard	9.8900

Detailed Cost Preparation of Foundation Course—	Cost per sq. yd.
Labor—	
Item:	
Scarifying	$0.0746
Shaping	0.1166
430 sq. yds. at	$0.1912

Concrete Base—	Cost per sq. yd.
Materials—	
Item:	
Crushed pebbles, 46.35 cu. yds., at $1.9375	$0.2089
Sand, 22.9 cu. yds., at $2.015	0.1073
Portland cement, 48.25 bbls., at $1.71	0.1917
Forms, etc., 150 ft. B. M., at $32.00	0.0112
Labor	0.3567
430 sq. yds. at	$0.8758

Concrete Curb—	Cost per sq. yd.
Materials—	
Item:	
Crushed pebbles, 7.18 cu. yds., at $1.9375	$0.0324
Sand, 3.6 cu. yds., at $2.015	0.0169
Portland cement, 7.5 bbls., at $1.71	0.0296
Forms, etc., 300 ft. B. M., at $32.00	0.0223
Labor	0.0690
430 sq. yds. at	$0.1702

Sand Cushion—	Cost per sq. yd.
Materials:	
Sand, 23.9 cu. yds., at $2.015	$0.1119
Labor	0.0306
430 sq. yds. at	$0.1425

Vitrified Blocks—	Cost per sq. yd.
Vitrified blocks, 18,402 at $27.00, 430 sq. yds. at	$1.1557

Laying Vitrified Blocks—	Cost per sq. yd.
Labor laying and rolling 430 sq. yds. at	$0.1787

Grouting and Covering—	Cost per sq. yd.
Materials—	
Item:	
Sand, 7.6 cu. yds. at $2.015	$0.0356
Portland cement, 11.75 bbls., at $1.71	0.0467
Labor	0.0314
430 sq. yds. at	$0.1137

Trimming Shoulders—	Cost per sq. yd.
Labor 430 sq. yds. at	$0.0392

SECTION 19—STATION 133 TO 139. BITUMINOUS PAVEMENT, PENETRATION METHOD, TARVIA "X."

Foundation Course.—The foundation course was originally old waterbound macadam with a telford base. After cleaning and spiking the old macadam, 1½-in. trap rock was spread and rolled to bring the surface to a grade 3 ins. below the finished grade. Screenings were then added and the road rolled and puddled until thoroughly cemented. A 12-ton 3-wheel roller was used.

Wearing Surface.—Three ins. of 1½-in. trap rock was spread upon the foundation course and rolled. One and one-half gals. per square yard of Tarvia "X," heated in tar kettles to a temperature of 230°, was then applied by a pressure distributor and covered with clean ¾-in. trap rock chips and rolled with a 12-ton 3-wheel roller.

Seal Coat.—A seal coat of ½ gal. per square yard of Tarvia "X" was applied by a pressure distributor at a temperature of 225° and covered with clean ½-in. trap rock chips and rolled.

Cost—	Cost per sq. yd.
Item:	
Preparation of foundation course	$0.2787
Wearing surface	0.5108
Seal coat	0.2279
Trimming shoulders	0.0403
	$1.0577

Detailed Cost Preparation of Foundation Course—	
Materials—	Cost per sq. yd.
Item:	
1½-in. trap rock, 55 cu. yds., at $2.6325	$0.1341
Trap rock screenings, 8.6 cu. yds., at $2.6325	0.0210
Labor—	
Item:	
Scarifying	$0.0293
Shaping	0.0254
Spreading trap rock	0.0388
Rolling and watering	0.0301
1079.75 sq. yds. at	$0.2787
Wearing Surface—	
Materials—	Cost per sq. yd.
Item:	
1½-in. trap rock, 90 cu. yds., at $2.6325	$0.2194
¾-in. trap rock, 20 cu. yds. at $2.6325	0.0488
Tarvia "X," 1671 gals. at $0.085	0.1315
Labor—	
Item:	
Spreading 1½-in. trap rock	$0.0362
Rolling 1½-in. trap rock	0.0359
Heating Tarvia "X"	0.0084
Applying Tarvia "X"	0.0125
Spreading ¾-in. chips	0.0122
Rolling ¾-in. chips	0.0059
1079.75 sq. yds. at	$0.5108
Seal Coat—	
Materials—	Cost per sq. yd.
Item:	
½-in. trap rock, 24.8 cu. yds., at $2.6325	$0.0605
Tarvia "X," 591 gals. at $0.085	0.0465
Labor—	
Item:	
Heating Tarvia "X"	$0.0058
Applying Tarvia "X"	0.0136
Spreading ½-in. chips	0.0170
Rolling ½-in. chips	0.0845
1079.75 sq. yds. at	$0.2279
	Cost per sq. yd.
Trimming Shoulders—	
Labor 1079.75 sq. yds at	$0.0403

SECTION 20—STATION 139 TO 145. BITUMINOUS PAVEMENT, PENETRATION METHOD, TEXACO ROAD ASPHALT BINDER.

Foundation Course.—The foundation course was originally old waterbound macadam with a telford base. After cleaning and spiking the old macadam, 1½-in. trap rock was spread and rolled to bring the surface to a grade 3 ins. below the finished grade. Screenings were then added and the road rolled and puddled until thoroughly cemented. A 12-ton 3-wheel roller was used.

Wearing Surface.—Three ins. of 1½-in. trap rock was spread upon the foundation course and rolled. One and eight-tenths gals. per square yard of Texaco Road Asphalt Binder, heated in tar kettles to a temperature of 350°, was then applied by hand and covered with clean ¾-in. trap rock chips and rolled with a 12-ton 3-wheel roller.

Seal Coat.—A seal coat of 6/10 of a gal. per square yard of Texaco Asphalt Binder was applied by hand at a temperature of 350° and covered with clean ½-in. trap rock chips and rolled.

Cost—	Cost per sq. yd.
Item:	
Preparation of foundation course	$0.2678
Wearing surface	0.5580
Seal coat	0.1714
Trimming shoulders	0.0182
	$1.0154

Detailed Cost Preparation of Foundation Course—	
Materials—	Cost per sq. yd.
Item:	
1½-in. trap rock, 61.8 cu. yds., at $2.6325	$0.1339
Trap rock screenings, 9.5 cu. yds., at $2.6325	0.0205
Labor—	
Item:	
Scarifying	$0.0235
Shaping	0.0366
Spreading 1½-in. trap rock	0.0288
Rolling and watering	0.0245
1214.73 sq. yds. at	$0.2678
Wearing Surface—	
Materials—	Cost per sq. yd.
Item:	
1½-in. trap rock, 101.2 cu. yds., at $2.6325	$0.2193
¾-in. trap rock, 22.5 cu. yds., at $2.6325	0.0488
Texaco asphalt binder, 2261 gals., at $0.10525	0.1959
Labor—	
Item:	
Spreading 1½-in. trap rock	$0.0314
Rolling 1½-in. trap rock	0.0114
Heating Texaco asphalt binder	0.0189
Applying Texaco asphalt binder	0.0128
Spreading ¾-in. chips	0.0087
Rolling ¾-in. chips	0.0108
1214.73 sq. yds. at	$0.5580
Seal Coat—	
Materials—	Cost per sq. yd.
Item:	
½-in. trap rock, 27.9 cu. yds., at $2.6325	$0.0605
Texaco asphalt binder, 730 gals., at $0.10525	0.0632
Labor—	
Item:	
Heating Texaco asphalt binder	$0.0069
Applying Texaco asphalt binder	0.0046
Spreading ½-in. chips	0.0082
Rolling ½-in. chips	0.0280
1214.73 sq. yds. at	$0.1714
	Cost per sq. yd.
Trimming Shoulders—	
Labor 1214.73 sq. yds. at	$0.0182

SECTION 21—STATION 145 TO 151. BITUMINOUS MACADAM PENETRATION METHOD, STANDARD BINDER B.

Foundation Course.—The foundation course was originally old waterbound macadam with a telford base. After cleaning and spiking the old macadam, 1½-in. trap rock was spread and rolled to bring the surface to a grade 3 ins. below the finished grade. Screenings were then added and the road rolled and puddled until thoroughly cemented. A 12-ton 3-wheel roller was used.

Wearing Surface.—Three inches of 1½-in. trap rock was spread upon the foundation course and rolled. One and one-half gals. per square yard of Standard Binder B, heated in tar kettles to a temperature of 350°, was then applied by hand, and covered with clean ¾-in. trap rock chips and rolled with a 12-ton 3 wheel roller.

Seal Coat.—A seal coat of ½ gal. per square yard of Standard Binder B was applied by hand at a temperature of 350° and covered with clean ½-in. trap rock chips and rolled.

Cost—	Cost per sq. yd.
Item:	
Preparation of foundation course	$0.3961
Wearing surface	0.4861
Seal coat	0.1442
Trimming shoulders	0.0516
	$1.0780

Detailed Cost Preparation of Foundation Course—	
Materials	Cost per sq. yd.
Items	
1½-in. trap rock, [illegible] cu. yds., at $2.6325	[illegible]
Trap rock screenings, 10[illegible] cu. yds., at $2.6325	[illegible]
Labor—	
Item:	
Scarifying	[illegible]
Shaping	[illegible]
Spreading 1½-in. trap rock	[illegible]
Rolling and watering	[illegible]
1085.84 sq. yds. at	$0.3961
Wearing Surface—	
Materials	Cost per sq. yd.
Item	
1½-in. trap rock, [illegible] cu. yds., at $2.6325	$0.2194
¾-in. trap rock, [illegible] cu. yds., at $2.6325	0.0487
Standard Binder B, 1630 gals., at $0.0905	0.1359
Labor—	
Item:	
Spreading 1½-in trap rock	$0.0235
Rolling 1½-in. trap rock	0.0104
Heating Standard binder	0.0164
Applying Standard binder	0.0125
Spreading ¾-in. chips	0.0089
Rolling ¾-in. chips	0.0104
1085.84 sq. yds. at	$0.4861
Seal Coat—	
Materials—	Cost per sq. yd.
Item:	
½-in. trap rock, 24.9 cu. yds., at $2.6325	$0.0604
Standard Binder B, 568 gals., at $0.0905	0.0474
Labor—	
Item:	
Heating Standard binder	$0.0065
Applying Standard binder	0.0045
Spreading ½-in. chips	0.0070
Rolling ½-in. chips	0.0184
1085.84 sq. yds. at	$0.1442
	Cost per sq. yd.
Trimming Shoulders—	
Labor 1085.84 sq. yds. at	$0.0516

SECTION 22—STATION 151 TO 157. BITUMINOUS PAVEMENT, PENETRATION METHOD, PIONEER ROAD ASPHALT.

Foundation Course.—The foundation course was originally old waterbound macadam with a telford base. After cleaning and spiking the old macadam, 1½-in. trap rock was spread and rolled to bring the surface to a grade 3 ins. below the finished grade. Screenings were then added and the road rolled and puddled until thoroughly cemented. A 12-ton 3-wheel roller was used.

Wearing Surface.—Three inches of 1½-in. trap rock was spread upon the foundation course and rolled. One and seven-tenths gals. per square yard of Pioneer Road Asphalt No. 963, heated in tar kettles to a temperature of 350°, was then applied by hand and covered with clean ¾-in. trap rock chips and rolled with a 12-ton 3-wheel roller.

Seal Coat.—A seal coat of ½ gal. per square yard of Pioneer Road Asphalt No. 963 was applied at a temperature of 350° and covered with clean ½-in. trap rock chips and rolled.

Cost—	Cost per sq. yd.
Item:	
Preparation of foundation course	$0.4549
Wearing surface	0.5475
Seal coat	0.1671
Trimming shoulders	0.0455
	$1.2150

Detailed Cost Preparation of Foundatoin Course—	
Materials—	Cost per sq. yd.
Items:	
1½-in. trap rock, 80.5 cu. yds., at $2.6325	$0.1976
Trap rock screenings, 12.4 cu. yds., at $2.6325	0.0304
Labor—	
Item:	
Scarifying	$0.0293
Shaping	0.1225
Spreading trap rock	0.0357
Rolling and watering	0.0394
1072.44 sq. yds. at	$0.4549
Wearing Surface—	
Materials—	Cost per sq. yd.
Item	
1½-in. trap rock, [illegible] cu. yds., at $2.6325	$0.2195
¾-in. trap rock, 19.9 cu. yds., at $2.6325	0.0489
Pioneer road asphalt No. 963, 1823 gals., at $0.10581	0.1799
Labor	
Item	
Spreading 1½-in. trap rock	$0.0373
Rolling 1½-in. trap rock	0.0123
Heating Pioneer Road Asphalt No. 963	0.0203
Applying Pioneer Road Asphalt No. 963	0.0109
Spreading ¾-in. chips	0.0082
Rolling ¾-in. chips	0.0102
1072.44 sq. yds. at	$0.5475
Seal Coat—	
Materials—	Cost per sq. yd.
Item	
½-in. trap rock, 21.6 cu. yds., at $2.6325	$0.0604
Pioneer Road Asphalt No. 963, 587 gals., at $0.10581	0.0579
Labor—	
Item:	
Heating Pioneer Road Asphalt No. 963	$0.0082
Applying Pioneer Road Asphalt No. 963	0.0044
Spreading ½-in. chips	0.0074
Rolling ½-in. chips	0.0288
1072.44 sq. yds. at	$0.1671
	Cost per sq. yd.
Trimming Shoulders—	
Labor 1072.44 sq. yds. at	$0.0455

SECTION 23—STATION 157 TO 163. BITUMINOUS PAVEMENT, PENETRATION METHOD, BERMUDEZ ASPHALT BINDER.

Foundation Course.—The foundation course was originally old waterbound macadam with a telford base.

After cleaning and spiking the old macadam, 1½-in. trap rock was spread and rolled to bring the surface to a grade 3 ins. below the finished grade. Screenings were then added and the road rolled and puddled until thoroughly cemented. A 12-ton 3-wheel roller was used.

Wearing Surface.—Three ins. of 1½-in. trap rock was spread upon the foundation course and rolled. One and eight-tenths gals. per square yard of Bermudez Asphalt Binder, heated in tar kettles to a temperature of 350°, was then applied by hand and covered with clean ¾-in. trap rock chips and rolled with a 12-ton 3-wheel roller.

Seal Coat.—A seal coat of six-tenths gal. per square yard of Bermudez Asphalt Binder was applied by hand at a temperature of 350° and covered with clean ½-in. trap rock chips and rolled.

Cost— Item:	Cost per sq. yd.
Preparation foundation course	$0.3571
Wearing surface	0.5932
Seal coat	0.1835
Trimming shoulders	0.0173
	$1.1511

Detailed Cost Preparation of Foundation Course Materials:	Cost per sq. yd.
1½-in. trap rock, 93 cu. yds., at $2.6325	$0.2137
Trap rock screenings, 14.3 cu. yds., at $2.6325	0.0329
Labor— Item:	
Scarifying	$0.0312
Shaping	0.0221
Spreading 1½-in. trap rock	0.0366
Rolling and watering	0.0206
1145.43 sq. yds. at	$0.3571

Wearing Surface— Materials— Item:	Cost per sq. yd.
1½-in. trap rock, 95.4 cu. yds., at $2.6325	$0.2193
¾-in. trap rock, 21.2 cu. yds., at $2.6325	0.0487
Bermudez Asphalt Binder, 2099 gals., at $0.1275	0.2336
Labor— Item:	
Spreading 1½-in. trap rock	$0.0342
Rolling 1½-in. trap rock	0.0167
Heating Bermudez Asphalt Binder	0.0128
Applying Bermudez Asphalt Binder	0.0097
Spreading ¾-in. chips	0.0078
Rolling ¾-in. chips	0.0104
1145.43 sq. yds. at	$0.5932

Seal Coat— Materials— Item:	Cost per sq. yd.
½-in. trap rock, 26.3 cu. yds., at $2.6325	$0.0604
Bermudez Asphalt Binder, 687 gals., at $0.1275	0.0765
Labor— Item:	
Heating Bermudez Asphalt Binder	$0.0043
Applying Bermudez Asphalt Binder	0.0032
Spreading ½-in. chips	0.0050
Rolling ½-in. chips	0.0333
1145.43 sq. yds. at	$0.1835

Trimming Shoulders—	Cost per sq. yd.
Labor, 1145.43 sq. yds., at	$0.0173

SECTION 24—STATION 163 TO 169. BITUMINOUS PAVEMENT, PENETRATION METHOD, SUN HYDROLENE ASPHALT.

Foundation Course.—The foundation course was originally old waterbound macadam with a telford base. After cleaning and spiking the old macadam, 1½-in. trap rock was spread and rolled to bring the surface to a grade 3 ins. below the finished grade. Screenings were then added and the road rolled and puddled until thoroughly cemented. A 12-ton 3-wheel roller was used.

Wearing Surface.—Three ins. of 1½-in. trap rock was spread upon the foundation course and rolled. One and seven-tenths gals. per square yard of Sun Hydrolene Asphalt Binder, heated in tar kettles to a temperature of 350°, was then applied by hand and covered with clean ¾-in. trap rock chips and rolled with a 12-ton 3-wheel roller

Seal Coat.—A seal coat of ½ gal. per square yard of Sun Hydrolene Asphalt Binder was applied by hand at a temperature of 350°, covered with clean ½-in. trap rock chips and rolled.

Cost— Item:	Cost per sq. yd.
Preparation of foundation course	$0.5419
Wearing surface	0.5383
Seal coat	0.1608
Trimming shoulders	0.0587
	$1.2997

Detailed Cost Preparation of Foundation Course— Materials— Item:	Cost per sq. yd.
1½-in. trap rock, 117.5 cu. yds., at $2.6325	$0.2884
Trap rock screenings, 18.1 cu. yds., at $2.6325	0.0444
Labor— Item:	
Scarifying	$0.0658
Shaping	0.0050
Spreading trap rock	0.0832
Rolling and watering	0.0551
1072.4 sq. yds. at	$0.5419

Wearing Surface— Materials— Item:	Cost per sq. yd.
1½-in. trap rock, 89.4 cu. yds., at $2.6325	$0.2194
¾-in. trap rock, 19.9 cu. yds., at $2.6325	0.0489
Sun Hydrolene Asphalt Binder, 1811 gals., at $0.0886	0.1496
Labor— Item:	
Spreading 1½-in. trap rock	$0.0467
Rolling 1½-in. trap rock	0.0032
Heating Sun Hydrolene Asphalt Binder	0.0266
Applying Sun Hydrolene Asphalt Binder	0.0129
Spreading ¾-in. chips	0.0104
Rolling ¾-in. chips	0.0206
1072.4 sq. yds. at	$0.5383

Seal Coat— Materials— Item:	Cost per sq. yd.
½-in. trap rock, 24.6 sq. yds., at $2.6325	$0.0604
Sun Hydrolene Asphalt Binder, 535 gals., at $0.0886	0.0442
Labor— Item:	
Heating Sun Hydrolene Asphalt Binder	$0.0074
Applying Sun Hydrolene Asphalt Binder	0.0045
Spreading ½-in. chips	0.0069
Rolling ½-in. chips	0.0374
1072.4 sq. yds. at	$0.1608
Trimming Shoulders— Labor, 1072.4 sq. yds., at	$0.0587

SECTION 25—STATION 169 TO 174 + 50. BICOMAC CONCRETE PAVEMENT.

Foundation Course.—The foundation course was originally old waterbound macadam with a telford base. This was removed to a depth of 5 ins. below the finished grade, and where the original surface was lower than this grade, the surface was loosened, and 1½-in. stone and screenings were spread and rolled to raise the surface to the grade required for the laying of the concrete base. A 5-ton tandem roller was used.

Concrete Pavement and Bituminous Top.—The concrete pavement, 4 ins. thick, and proportioned 1-3-6, was laid, and as soon as the concrete had settled, it was rolled with a hand roller weighing 700 lbs., and while still wet and in a plastic condition, was covered with a resilient concrete wearing surface 1-in. thick after compression. The wearing surface, or bituminous top, was applied cold, and consisted of 1 part Portland cement, 6 parts clean sand, and 12 parts clean trap rock passing a 1½-in. screen, and containing not more than 25 per cent of 1½-in. size stone, and 8 per cent of Headley Good Roads Company No. 26, known as Bicomac, which was used in the proportion of 70 per cent of Bicomac, and 30 per cent of water. The mineral aggregate was thoroughly coated with the solution. The sand and cement were mixed together and added to the coated stone and the whole thoroughly saturated with the solution and laid and rolled with a 5-ton tandem roller, as soon as the base had become hard enough to sustain the roller.

Cost— Item:	Cost per sq. yd.
Preparation of foundation course	$0.1461
Concrete pavement	0.9777
Bituminous top	0.6515
Trimming shoulders	0.0918
	$1.8671

Detailed Cost Preparation of Foundation Course— Labor— Item:	Cost per sq. yd.
Scarifying	$0.0306
Shaping	0.1155
1015 sq. yds. at	$0.1461

Concrete Pavement— Material— Item:	Cost per sq. yd.
Trap rock, 109.39 cu. yds., at $2.4705	$0.2690
Gravel, 54.13 cu. yds., at $1.82	0.0981
Portland cement, 113.25 bbls., at $1.65	0.1860
Coal, 6 tons, at $5	0.0299
Forms, etc., 750 ft. B. M., at $32	0.0239
Labor	0.3708
1,015 sq. yds. at	$0.9777

Bituminous Top Bicomac— Material: Item.	Cost per sq. yd.
Trap rock, 26.32 cu. yds. at $2.4705	$0.0647
Sand, 13.17 cu. yds. at $1.82	0.0239
Portland cement, 15 bbls. at $1.65	0.0246
Bicomac, 1,653.37 gals. at $0.10997	0.1810
Coal, 2 tons at $5	0.0400
Forms, etc., 350 ft. B. M. at $32	0.0112
Labor: Item.	
Mixing	0.2383
Applying	0.0794
Rolling	0.0184
1,015 sq. yds. at	$0.6515

Trimming Shoulders— Labor:	Cost per sq. yd.
1,015 sq. yds. at	$0.0918

SECTION 26—STATION 174 + 50 TO 180 + 15. VITRIFIED BLOCK PAVEMENT ON 4-IN. CONCRETE BASE.

Foundation Course.—The foundation course was originally old waterbound macadam with a telford base. The surface was graded to 9½ ins. below the finished grade and additional material was supplied where needed and rolled.

Concrete Base and Vitrified Blocks.—A 4-in. concrete base with concrete curb, proportioned 1-3-6, was laid upon the prepared subgrade. The concrete was mixed in a Foote Concrete Mixer and by hand labor.

The sand cushion was spread, rolled with an iron hand roller and shaped with a template. After laying, the vitrified blocks were rolled with a 4-ton tandem roller, the joints were grouted to within ½ in. of the surface with cement grout, proportioned 1-1, and covered with sand.

Cost— Item:	Cost per sq. yd.
Preparation of foundation course	$0.1038
Concrete base	1.0806
Concrete curb	0.1991
Sand cushion	0.1335
Vitrified blocks	0.9631
Laying vitrified blocks	0.1635
Grouting and covering	0.1175
Trimming shoulders	0.1223
	$2.8834
Concrete base, per cu. yd	$9.829
Concrete curb, per cu. yd	12.2290
Concrete curb, per lin. ft	0.1790

Detailed Cost, Preparation of Foundation Course— Labor— Item:	Cost per sq. yd.
Scarifying	$0.0273
Shaping	0.0763
	$0.1038

Concrete Base— Materials: Item.	Cost per sq. yd.
Trap rock, 109.39 cu. yds. at $2.4705	$0.2662
Sand, 53.56 cu. yds. at $1.82	0.0960
Portland cement, 112.25 bbls. at $1.65	0.1825
Forms, etc., 500 ft. B. M. at $32	0.0158
Coal, 5 tons at $5	0.0246
Labor	0.4955
1,004.4 sq. yds. at	$1.0806

Concrete Curb— Materials: Item.	Cost per sq. yd.
Trap rock, 16.03 cu. yds. at $2.4705	$0.0390
Sand, 7.93 cu. yds. at $1.82	0.0142
Portland cement, 16.75 bbls. at $1.65	0.0272
Forms, etc., 750 ft. B. M. at $32	0.0236
Labor	0.0951
1,004.4 sq. yds. at	$0.1991

Sand Cushion— Materials:	Cost per sq. yd.
Sand, 55.8 cu. yds. at $1.82	$0.1001
Labor	0.0334
1,001.4 sq. yds. at	$0.1335

Vitrified Blocks— Materials:	Cost per sq. yd.
Vitrified blocks, 39,473, at $24.654 per M.	$0.9631
1,004.4 sq. yds. at	0.9631

Laying Vitrified Blocks— Labor:	Cost per sq. yd
Laying and rolling, 1,004.4 sq. yds.	$0.1635

Grouting and Covering—

Materials: Item.	Cost per sq. yd.
Portland cement, 27.75 bbls. at $1.65	$0.0456
Sand, 17.87 cu. yds. at $1.82	0.0324
Forms, etc., 100 ft. B. M. at $32	0.0032
Labor	0.0363
1,004.4 sq. yds. at	$0.1175

Trimming Shoulders— Labor:	Cost per sq. yd.
1,004.4 sq. yds. at	$0.1223

SCHEDULE OF WAGES.

1912.		Per Day.
Superintendents, $125 per mo.	10 hours	$4.81
Foremen	10 hours	3.50
Carpenters	8 hours	3.60
Roller engineers	10 hours	3.00
Paviors (straight time men)	10 hours	2.50
Watchmen	12 hours	1.75
Watchmen (after Nov. 6, 1912)	12 hours	2.00
Laborers	10 hours	1.75
Laborers (after Nov. 6, 1912)	10 hours	2.00
Laborers (asphalt plant)	10 hours	2.25
Single teams	10 hours	3.00
Double teams	10 hours	5.00
Double teams (aft. Nov. 6, 1912)	10 hours	6.00

1913.		Per Day.
Foremen	10 hours	$3.50
Foremen	9½ hours	3.50
Foremen	10 hours	3.00
Laborers	10 hours	2.00
Laborers	9½ hours	2.15
Laborers	9 hours	2.00
Water boys	9½ hours	1.00
Single teams	9½ hours	3.00
Double teams	9½ and 10 hours	5.50
Paviors	8 hours	4.50
Rammers	8 hours	4.00

SCHEDULE OF CHARGES FOR TOOLS, ETC.

Rollers, 12-ton, 3-wheel, with engr., etc.	$12.00
Rollers, 6-ton tandem, with engineer	8.00
Rollers, 4-ton tandem, with engineer	10.00
Water wagons	.50
Tar kettles	.50

No allowance was made for the use of the asphalt plant on mixing sections.

A Sub-Organization for Securing Efficient Highway Maintenance.

A complete reorganization of the New York State Highway Department, recommended by a board of consulting engineers, has been adopted by Mr. John N. Carlisle, Commissioner of Highways, and the names of division engineers appointed in accordance with the plans for reorganization were published in our issues of Oct. 29 and Nov. 5. An important feature of the reorganization consists of the consolidation of construction and maintenance forces into one organization, similar to the arrangement of the engineering and maintenance of way departments on many railroads. In a paper presented to the tenth annual convention of the American Road Builders' Association held at Philadelphia, Dec. 9-12, 1913, Mr. Carlisle explains how the new organization will be made and how it will handle the maintenance work. The paper in full follows:

The really great problem confronting those who are in charge of state departments of highways is maintenance. This question involves not only a study of the methods to be adopted for properly keeping the roads in repair, but first involves the question of what scheme of organizing a department can be installed to procure such a result.

In New York state up to this year the construction of highways was under the charge of division engineers, and the maintenance work was separate and apart and was in charge of a maintenance bureau under different divisions and under superintendents of maintenance. This plan of divided authority was so unsatisfactory that in the winter of 1913 the state legislature of New York provided that the work of construction and maintenance should be consolidated under the control of the division engineers, and that the work of maintenance should be done either by contract or directly by the department.

New York is divided into 9 divisions with a division engineer in charge of each division, and we intend to subdivide each of these divisions into about 7 subdivisions, making 63 subdivisions of the state. In charge of each of these subdivisions we intend to appoint an assistant engineer, and put him in charge of both construction and maintenance in his territory, requiring him to have an office and live therein, and to hold him rigidly responsible for the condition of the roads, in his territory.

The New York plan in the past has been to rely largely upon patrolmen, that is the placing of a man with a horse and wagon in charge of a short number of miles, and imposing upon him the duty of keeping his section of the road in repair. With a state so large as New York, this plan has not worked out to our satisfaction, and while we do not believe in entirely abandoning the patrol system, yet we are arranging for an organization of section gangs in each of the subdivisions, each section gang to be furnished with equipment with which roads can be oiled and repaired.

No railroad company today would think of trying to keep its tracks in repair simply by the patrol system, and we believe that in following the methods of the steam railroads in taking care of their tracks we can accomplish better results than in any other way. This problem, of course, means the organization of a proper section gang with equipment sufficient to do repair work. Our plans call for the furnishing of an auto truck, a pressure sprayer oiler, a steam roller, and heating tanks, and the necessary small tools for each section, and each section gang will be large enough and well enough equipped to properly repair any type of a road already constructed.

With section gangs properly organized, and with sufficient equipment under the direction of an engineer who is held responsible, we ought to produce a sub-organization which can give efficient maintenance. Section gangs will be organized strong enough to clean and oil the roads at the proper season, make temporary repairs, and when this preliminary work is done, make heavy permanent repairs to the roads.

A New Gasoline Tractor with Dust-Proof Running Parts.

(Staff Article.)

One cause of frequent repair bills for all machinery used for work on fields or haulage is the continual grind from dust and dirt on gearing and running parts. All working parts of the tractor illustrated by the accompanying cut, where practicable, are enclosed and run in oil. The makers claim the type of protection used to be dust-proof. A 14-in. belt pulley driven from the motor by gears is provided for use when it is desired to run machinery with the motor.

The tractor is mounted on helical springs, and is equipped with a spring draw bar. Some of the advantages claimed for the Wallis Tractor by the manufacturers are as follows: Light weight; dust-proof construction; three-speed transmission with only three reductions for forward speeds with the consequent saving in power; liberal use of high grade roller bearings; motor with but few and simple parts; ability of machine to turn around in a space having a diameter equal to its wheelbase; economy in fuel consumption; and simple and strong construction.

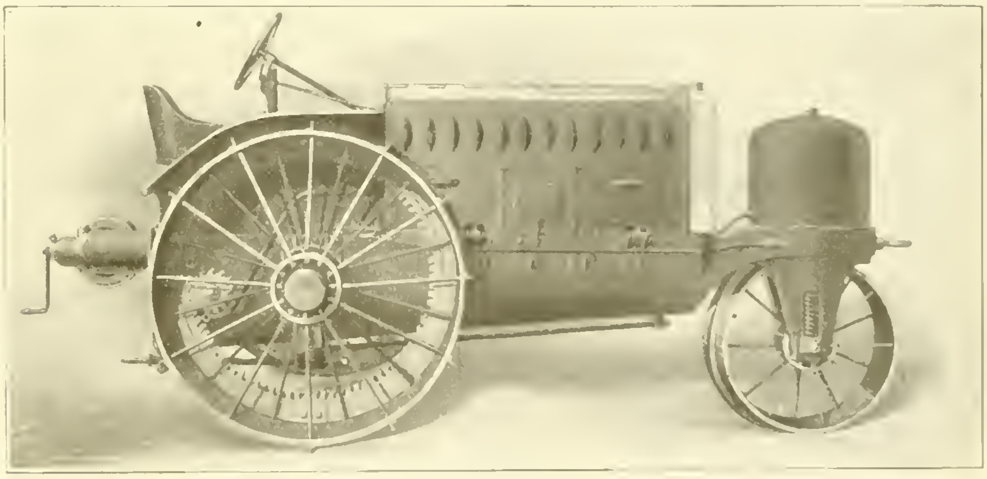


Fig. 1. A Three-Wheeled Gasoline Tractor for General Hauling Purposes.

The tractor weighs 7,500 lbs., and its general dimensions are: Length over all, 14 ft.; width over all, 6 ft. 2 ins.; height over all, 6 ft., and wheel base, 8 ft. 4 ins. A 4-cylinder motor, capable of developing 25 B. H. P. at 600 R. P. M. furnishes power for the machine, which is rated as having 15 H. P. at the drawbar. A speed of from 2 to 3½ miles per hour is obtained with the motor running at normal rate. This tractor is manufactured by the Wallis Tractor Co., Racine, Wis.

Railways

A Comparison of Results of Different Methods Used in Computing Earthwork Overhaul.

(Staff Article.)

A comparison of the results obtained by computing overhaul in earthwork problems by some of the various methods in use has been prepared by Mr. Frank Gahrtz, Civil Engineer, Oklahoma City, Okla. As the results of no two methods are alike, in his comparison, for a copy of which we are indebted to Mr. Walter H. Meier, Civil Engineer, Oklahoma City, our readers may be interested in studying the methods compared and the results obtained. The following definitions of terms used in overhaul problems are used in the comparison.

Overhaul. The distance exceeding a stated amount of distance that material is hauled in constructing a roadbed. *Free Haul*, the distance within a stated amount (the stated amount referred to in the definition of overhaul) that material is hauled in constructing a roadbed. The limits of free haul shall be determined by fixing on the profile two points —one on each side of the neutral grade point —one in excavation and the other in embankment, such that the distance between them shall equal the specified free haul, and the included quantities of excavation and embankment, after making allowance for shrinkage or swell, balance. *Free Haul Mass*, the quantity of material in excavation between the limits of free haul. *Overhaul Masses*, the quantities of material in excavation and resulting embankment exclusive of the free haul mass. *Units of Overhaul,* 1 cu. yd. hauled 100 ft. is the unit by which payment for overhaul shall be made.

METHOD I, GIVEN IN MOLITOR AND BEARD'S "MANUAL FOR RESIDENT ENGINEERS."

Molitor and Beard in their "Manual for Resident Engineers," obtain the overhaul distance by deducting the free-haul distance from the distance between the centers of mass of excavation and embankment, the latter distance being obtained graphically by means of a "mass diagram." They obtain the units of overhaul or "the number of pay cubic yards hauled in 100 ft.," by mutiplying the number of cubic yards hauled by the length of overhaul in 100-ft. stations. This includes the free-haul mass, and this method is therefore greatly in favor of the railway company, and an injustice to the contractor, as will be shown in the comparison of results following the discriptions of the methods compared.

METHOD II, GIVEN IN AMERICAN RAILWAY ENGINEERING ASSOCIATION MANUAL.

This method is in use by a number of engineers who after separating the free-haul mass, which receives no further consideration, treat with the overhaul masses only, computing the overhaul by subtracting the free haul from the distance between the centers of the overhaul masses. The number of units of overhaul are computed by multiplying the length of overhaul in 100-ft. stations by the number of cubic yards in the overhaul mass.

The unfairness of this method in final estimates to contractors is illustrated in Fig. 1 by the dotted profile and dotted section of the mass diagram, which shows a case where the total overhaul mass (5,200 cu. yds.) and the center of overhaul mass remain the same as in the example used for comparison of methods, but the haul distance becomes greater. The effect on the overhaul distance and number of units of overhaul as computed by Method III for this increase in length of haul is given in the comparison under the heading Special Case by Method III.

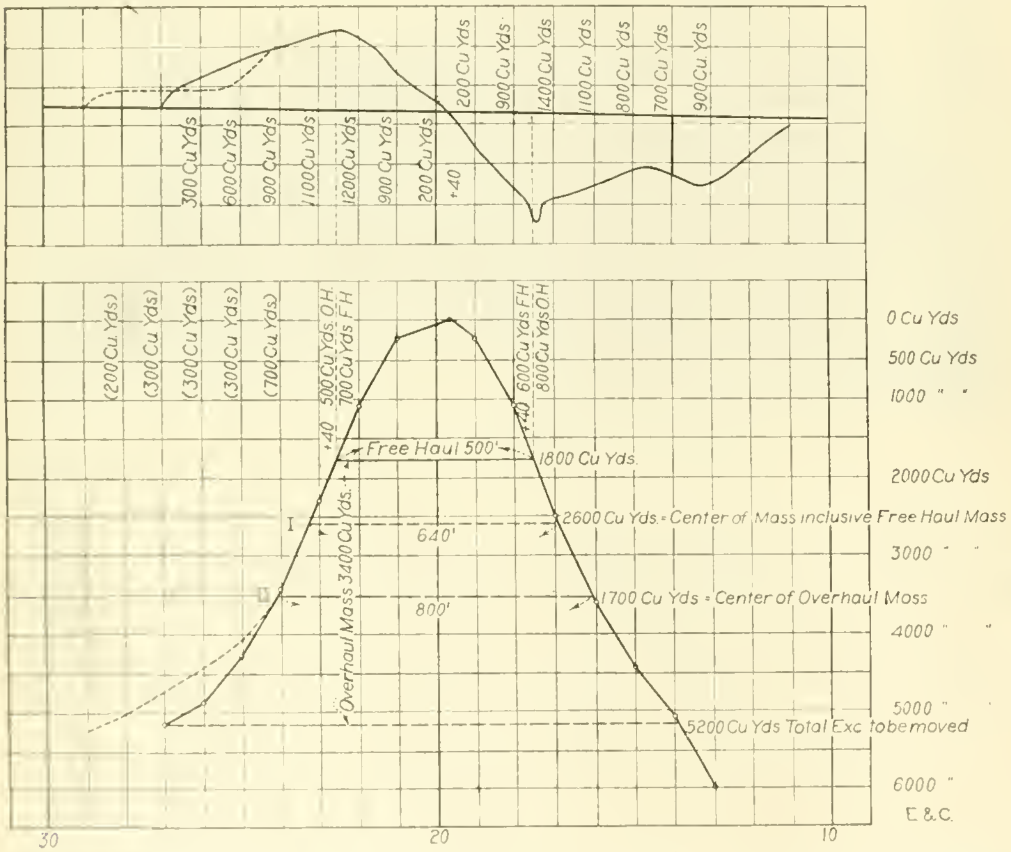


Fig. 1. Profile and Mass Diagram Used in Comparison of Methods of Computing Overhaul.

METHOD III—METHOD SUGGESTED BY MR. GAHRTZ.

Establish the ends of the free haul (say 500 ft.) graphically, then proceed with the use of prisms extending from one cross section to another as units, multiplying the volume of each such prism in the overhaul masses of excavation and embankment by the distance in feet between its center of mass and the proper adjoining limit of free haul. Add the excavation items and embankment items in two separate amounts. The sum of these two amounts divided by the volume in the overhaul mass of excavation will be the overhaul distance in feet. The volume (in cubic yards) in the overhaul mass of excavation multiplied by the overhaul distance in 100-ft. stations, gives the number of units of overhaul.

While Method II is applicable to approximate estimates, Method III should be used in preparing final estimates to do justice to contractors as well as railway companies.

At the top of Fig. 1 is shown a short section of profile, the number of cubic yards of excavation or embankment in each 100-ft. station being indicated thereon. In the mass diagram below the profile the graphical determination of the distance (640 ft.) between the centers of mass according to Method I, is indicated by the horizontal line marked I. In like manner the graphical determination of the distance (800 ft.) between the centers of the overhaul masses used in Method II is designated by the horizontal line marked II. It might be well to state that the plotting of the mass diagram was done by the usual method of making such diagrams. As mentioned under the description of Method II, the dotted lines on the profile and mass diagram represent change of profile, but not of total quantity of excavation which affects the overhaul distance and number of units of overhaul. The earthwork quantities in parentheses, under the left-hand end of the cut, indicate the change in yardage in the 100-ft. stations. The numerical part of the comparison follows, 500 ft. being the free haul in all cases:

METHOD I.

$$5{,}200 \text{ cu. yds.} \times \left(\frac{640 \text{ ft.} - 500 \text{ ft.}}{100 \text{ ft.}}\right) = 7{,}280 \text{ units}$$

of overhaul.

METHOD II.

The location of the free-haul limits, obtained graphically, is indicated in Fig. 1. There are 1,800 cu. yds. of excavation within these limits; 5,200 cu. yds. — 1,800 cu. yds. = 3,400 cu. yds.

$$3{,}400 \text{ cu. yds.} \times \left(\frac{800 \text{ ft.} - 500 \text{ ft.}}{100 \text{ ft.}}\right) = 10{,}200 \text{ units}$$

of overhaul.

METHOD III.

The free-haul limits coming at plus stations, cause the quantities between stations 22 and 23, and between stations 17 and 18 to be divided; part of each 100 ft. prism falling within the limits of free haul; 500 cu. yds. of excavation between stations 22 and 23 being left as part of the overhaul mass in excavation; and 700 cu yds. of embankment being in a similar way part of the overhaul mass in embankment, Table I.

TABLE I.

Cu. yds.	Ft.		Cu. yds.	Ft.	
500 ×	20 =	10,000	800 ×	30 =	24,000
1,100 ×	90 =	99,000	1,100 ×	110 =	121,000
900 ×	190 =	171,000	800 ×	210 =	168,000
600 ×	290 =	174,000	700 ×	310 =	217,000
300 ×	390 =	117,000			
3,400		571,000	3,400		530,000

$$\frac{571{,}000 + 530{,}000}{3{,}400} = 323.8 \text{ ft.} = \text{overhaul distance.}$$

$$\frac{571{,}000 + 530{,}000}{100} = 11{,}010 \text{ units of overhaul.}$$

SPECIAL CASE BY METHOD III.

In all cases "swell" or "shrinkage" of material as well as the material from ditches in the cuts and on the surface should be taken into account. These items have been omitted from this comparison in order to simplify the computations and explanation.

TABLE II.

Cu. yds.	Ft.		Cu. yds.	Ft.	
500 ×	20 =	10,000	800 ×	30 =	24,000
1,100 ×	90 =	99,000	1,100 ×	110 =	121,000
700 ×	190 =	133,000	800 ×	210 =	168,000
300 ×	290 =	87,000	700 ×	310 =	217,000
300 ×	390 =	117,000	3,400		530,000
300 ×	490 =	147,000			
200 ×	590 =	118,000			
3,400		711,000			

$$\frac{711{,}000 + 530{,}000}{3{,}400} = 382.6 \text{ ft.} = \text{overhaul distance.}$$

$$\frac{711{,}000 + 530{,}000}{100} = 12{,}410 \text{ units of overhaul.}$$

Methods and Costs of Painting An Interurban Railway Car, Green Bay, Wis.

(Staff Article.)

A one-compartment interurban car belonging to the Wisconsin Public Service Co. of Green Bay, Wis., was painted in the company's shops at a cost of $81.50.

After the loose paint was removed, the car, which had double trucks and was 40 ft. long over all, was given five coats of paint in the following order: One coat of Tector, two coats of color varnish, and two coats of outside varnish. Two coats of varnish were put on the interior of the car, two coats of paint on the floor, one coat of varnish on the seats, and a coat of roof paint on the roof. Only the window guards were removed from the car during the work of painting, the doors and windows being left in place. The work was all done by two painters between Oct. 1 and Oct. 26, 1913, inclusive, no carpenter work being necessary. A one-color process of painting was used and the trim was red. Ironhide paint was applied to the trucks. The low unit cost of labor for painting was only possible on account of the fact that the men who did the work are employed the year round and do other kinds of work when no painting is to be done. The cost of materials and labor for painting the car are given in the following tabulation:

COST OF PAINTING A 40-FT. INTERURBAN CAR.

Materials—	Cost.
8 gals. dry white lead	$ 0.58
1 can road cart red	1.26
1 gal. turpentine	0.55
1 gal. Tector	1.50
3 gals. floor paint	3.72
2 gals. Ironhide paint	2.46
1½ gals. dark color varnish	3.33
2 gals. outside varnish	1.88
½ gal. spar varnish	1.34
48 sheets sand paper	0.48
Total cost materials	$20.10
Labor—	
307 hours' painting at 20 cts.	$61.40
Total cost	$81.50

Cost Data on Lining Two Tunnels.

Contributed by George Harper, New York City.

The accompanying data give the cost of lining two tunnels, exclusive of the portals. The tunnel of 4,211 ft. length is located in West Virginia and the 4,000 ft. tunnel is in Pennsylvania. The West Virginia figures are for brick arching and packing alone. On the Pennsylvania tunnel the entire lining from footings up is inclusive of one course of fire brick lining in arch area affected by gas and smoke. This single course lining was tied into the concrete with headers every fourth course.

TABLE I.—COST OF BRICK ARCH TUNNEL LINING (4,211 LIN. FT.) IN WEST VIRGINIA TUNNEL.

	Total shifts.	Total cost.	Cost per lin. ft.
Bricklaying	7,392	$ 20,580	$ 4.84
Moving centers	672	1,806	0.43
Lagging	504	1,176	0.26
Packing	2,520	4,410	1.05
Key	840	1,075	0.26
Material, delivery from west portal	1,176	3,570	0.85
Air service and pipe line	168	6,093	1.45
Center erection and dismantling	168	3,000	0.71
Mixer, erection, etc.	168	600	0.14
Tracks and switches	168	500	0.12
Lighting, oil, etc.	168	1,600	0.38
Timekeeper, office and plant	168	2,430	0.55
Brick (5,396,000)	168	44,517	10.57
Cement (6,600 bbls.)	168	9,240	2.20
Sand (3,500 cu. yds.)	168	2,800	0.66
Packing stone (9,050 cu. yds.)	168	6,787	1.61
Extra brick (portal and bad ground)		292	0.07
Total cost		$110,476	$26.21

The West Virginia work was put up by thoroughly experienced tunnel contractors, whose system, organization and general methods were of the best and had been evolved from the result of years of experience. The work on the tunnel in Pennsylvania was done by another firm with not so much experience in this line of work, and whose methods were somewhat more expensive, cumbersome and inexperienced. The costs are from private notes and from close contact with both of these undertakings. The brick lining cost in the 4,211-ft. tunnel in West Virginia is $26.15 per lin. ft., for the arching and packing, and with the concrete walls and footings added it will approximate in all about $40.15 per lin. ft.

TABLE IV.—COST OF MATERIAL FOR ARCH AND PACKING OF WEST VIRGINIA TUNNEL, F. O. B. PORTAL.

	Rate.	Total	Approx. cost per lin. ft.
Brick (5,396,000 M.)	$8.25	$ 44,517	$10.57
Cement (6,600 bbls.)	1.40	9,240	2.20
Sand (3,500 cu. yds.)	.80	2,800	0.66
Packing stone (9,050 cu. yds.)	.75	6,787	1.61
		$110,184	$26.14

TABLE II.—COST DETAILS FOR 4,211 FT. OF ARCHING IN WEST VIRGINIA TUNNEL.

	Rate.	No. men per shift.	No. shifts per man in tots.	Total No. of shifts of 25 lin. ft. each.	Cost of shifts.	Approx. cost per lin. ft.
Bricklaying:						
General foreman	$6.00	1	168	168	$ 1,088	$0.25
Foreman bricklayer	9.00	1	168	168	1,512	0.36
Bricklayers	8.00	4	168	672	5,376	1.29
Labor foreman	4.00	1	168	168	672	0.16
Mortar tenders	2.25	5	168	840	1,890	0.45
Mortar mixer	2.25	1	168	168	378	0.09
Mortar laborers	2.00	3	168	504	1,008	0.24
Mortar box	2.00	1	168	168	336	0.08
Mortar laborers	2.00	3	168	504	1,008	0.27
Mortar mule and drivers	3.50	1	168	168	588	0.14
Mortar hoister and runner	2.00	1	168	168	336	0.08
Brick laborers	1.75	20	168	3,360	5,880	1.37
Water boy	1.75	1	168	168	294	0.07
Light and lamp tenders	1.75	1	168	168	294	0.07
Total		44		7,392	$20,580	$4.84
Moving Centers:						
Foreman carpenters	$4.00	1	168	168	$ 672	$0.16
Laborers	2.25	3	168	504	1,134	0.27
Total		4		672	$ 1,806	$0.43
Lagging:						
Carpenter	$3.00	1	168	168	$ 504	$0.11
Laborers	2.00	2	168	336	672	0.15
Total		3		504	$ 1,176	$0.26
Packing:						
Laborers	$1.75	15	168	2,520	$ 4,410	$1.05
Key:						
Bricklayer	$4.00	1	168	168	$ 672	$0.16
Laborers	.60	4	168	672	403	0.10
Total		5		840	$ 1,075	$0.26

TABLE III.—MISCELLANEOUS COST DATA ON WEST VIRGINIA TUNNEL.

Material, Delivery from W. Portal to Work.	Rate.	No. men per shift.	No. shifts per man in tots.	Total No. of shifts of 25 lin. ft. each.	Total.	Approx. cost per lin. ft.
Dinkeys	$ 5.00	2	168	336	$1,680	$0.40
Dinkey engineers	2.75	2	168	336	924	0.22
Dinkey brakemen	2.00	2	168	336	672	0.16
Dinkey watchmen	1.75	1	168	168	....	0.07
				1,176	$3,570	$0.85
Air Service:						
Hoisting, mixing, etc.	36.32		168	168	$6,092	$1.45
Center erection and dismantling	1,000.00	3		168	3,000	0.71
Mixer erection and moving	200.00	3		168	600	0.14
Tracks and Switches:						
Propn. charges	500.00			168	500	0.12
Lighting and Oils:						
9,000 gals. gasoline at 17 cts.	.17			168	1,530	0.36
200 gals. kerosene	.15			168	30	0.01
400 gals. blackoil (centers)	.10			168	40	0.01
Interest on Plant, Etc.:						
Arbitrary	750.00			168	750	0.18
Propn. timekeeper and office	10.00			168	1,680	0.38

WEST VIRGINIA TUNNEL.

As stated the length of this tunnel is 4,211 ft. The tunnel width is 31 ft. The radius of the arch is 15 ft. 6 ins. The arch consists of five rings of brick laid up in five courses with 1 to 3 mortar. An individual brick measured 3x4x9 ins. The bricklayers worked 167 shifts. Only one shift was worked by others than bricklayers. The time lost due to moving, delays, etc., amounted to 28 shifts. The number of shifts as calendar days amounted to 196. The average length of tunnel lined per shift for the 168 shifts of actual working time was 25 ft. The tunnel lining was carried on simultaneously at different points along the length of the tunnel and for this reason four closures were made. For each lin. ft. of arch 1,227 bricks were required, making in all 5,166,897 bricks in the arch. In each spandrel wall, spacing 5 ft. 4 ins., there were 1,561 bricks; 156,400 bricks in all were used for the spandrel walls. For extra work including portals, bad ground, etc., 31,825 bricks were used. The number of culls, bats, etc., amounting to ¾ of 1 per cent, were 10,878, making the total number of bricks used for the lining 5,396,000. The packing averaged 2 cu. yds. and 1 cu. ft. to each lineal

TABLE V.—COST DATA ON LINING AND ARCHING ON 4,000 LIN. FT., AND WITHOUT PORTALS, PENNSYLVANIA TUNNEL.

Cost, Walls, Footings and Arches	Amounts.	cu. yd	lin. ft	Basis of Figures.
Arches, walls with footings and brick	$216,794	$7.74	$54.20	154.0[illegible] cu. yds. concrete in walls.
Arches, walls with brick and without footings	207,909	7.43	51.98	
Arches, walls with footings and without brick	196,041	7.00	49.01	120.41 cu. yds. concrete in arch.
Arches, walls without brick and footings	187,119	6.68	46.79	
Footings:				11.08 cu. yds. concrete in footings
Material and labor, 8,000 ft. = 1,777 cu. yds. at $	8,885		2.22	
Bricklaying and Labor:				
On 4,000 ft. arch only	20,760		5.19	286.07 cu. yds. concrete in sec. of 40 ft.
				28,000 cu. yds. concrete, cement and brick used as concrete unit, and without footings.

ESTIMATED QUANTITIES OF MATERIAL

Cement, bbls., 31,500 "Alpha"	@ $ 1.10 per bbl.	Does not include footings.
Sand, cu. yds., 12,160	@ 1.40 per c. y.	
Gravel, cu. yds., 11,[illegible]60	@ 1.20 per c. y.	
Stone, cu. yds., 7,[illegible]80	@ 1.30 per c. y.	
Fire brick, M's 900	@ 11.50 per M	

Work con[illegible] 16[illegible] days, an average of [illegible] ft. per day, inclusive of all delays.
Proportion used mortar 1 [illegible]
Proportion used Concrete 1 [illegible]

TABLE VI.—COST OF PENNSYLVANIA TUNNEL LINING, CONCRETE WALLS AND ARCHING, FOR ENGINES, CARS AND LABOR ONLY.
(4,000 lin ft.)

	Av. No. of men, basis of 13-hour shift.	Unit of time for each 40' sec. 39 hours.	Rate per hour.	Cost per 39-hour unit on each 40' sec.	Proportion of cost per lin. ft.
Walls, Bricklaying and Arching.					
Arching—Bricklayers	1	39	$1.00	$ 39.00	$0.98
Foremen	1	39	.40	15.60	0.39
Labor	10	390	.20	78.00	1.95
Carpenters	1	39	.35	13.65	0.34
Walls—Foreman, ppn.	½	39	.40	7.80	0.20
Labor	6	234	.20	46.80	1.17
Bulkhead carpenter	1	39	.35	13.65	0.34
Forms—Foreman, ppn.	½	39	.40	7.80	0.20
Labor, ppn.	2	78	.25	19.50	0.49
Walls and Arching:					
Hoister runner	1	39	.25	9.75	0.24
Dinkey engines	3	117	.50	58.50	1.46
Dinkey engineers	3	117	.27½	32.17	0.80
Dinkey brakemen	3	117	.20	23.40	0.58
Cement and brick cars	9	351	.04	14.04	0.35
				$379.66	$9.49

TABLE VII.—COST OF PENNSYLVANIA TUNNEL LINING FOR MATERIAL, PLANT, LIGHTING AND OVERHEAD CHARGES.
(4,000 lin. ft.)

Estimated Costs.	Cost per cu. yd.	Cost per lin. ft.
Cement, sand, gravel and stone	$2.9258	$20.48
Brick, 900 M	0.3750	2.63
Blaw forms	0.3214	2.25
Blaw forms, erection and dismantling (4)	0.2760	1.93
Plant E. portal, boilers, machy., storage bins W. H., etc., erection and dismantling	0.5357	3.75
Electric and gasoline lighting	0.1690	1.18
Superintendence, 5½ months	0.0535	0.38
Office, etc., 5½ months	0.1236	0.87
Air and pipe lines, $50 per day for 173 days	0.3089	2.16
Tracks, switches, etc	0.0535	0.37
Interest and incidentals (estimated)	0.0893	0.62
Stock, cement, etc., mixer and labor	0.8384	5.87
Walls and arching, labor and machinery	1.3551	9.49
8,000 lin. ft. footings	0.3173	2.22
Total cost	$7.7420	$54.20

TABLE VIII.—COST OF PENNSYLVANIA TUNNEL LINING, STOCK PIPE LINES, CEMENT, WAREHOUSE AND MIXER, LABOR ONLY.
(4,000 lin. ft.)

Warehouse, Material and Mixing Stock, Pipe Line, Etc.	Av. No. of men on basis of 13-hour shift.	Unit of time for each 40' sec. 39 hours.	Rate per hour.	Ppn. of cost per lin. ft.
Industrial crane engr., ppn.	1	13	$0.35	$0.11
Industrial crane fireman, ppn.	1	13	0.25	0.08
Industrial crane watchman, ppn.	1	13	0.25	0.08
General foreman	1	26	0.50	0.32
Foreman	1	39	0.40	0.39
Laborers	12	468	0.20	2.34
Incline runner	1	39	0.25	0.24
Mixer bin laborers	3	117	0.20	0.58
Cement W. St. laborers	2	78	0.20	0.39
Mixer laborers	3	117	0.20	0.59
Mixer engineer	1	39	0.25	0.24
Mixer fireman	1	39	0.25	0.24
Pump fireman	1	39	0.25	0.24
				$5.84

foot of tunnel. The cost data on the West Virginia work are given in Tables I to IV, inclusive. The cost data on the Pennsylvania tunnel are given in Tables V to VIII, inclusive. The former work was done in 1911 and 1912, while the latter was done in 1912, from July to December.

Unit Prices for Electric Current for Operating Urban Electric Lines, Chicago, Ill.

(Staff Article.)

All Chicago elevated and surface lines will by a new contract operate by purchased power alone. For some years the city's surface lines have bought all operating current, but the elevated lines have in part generated their own current. The new arrangement involves a new schedule of rates which is given by Table I. The scale of prices is made up of two parts, a primary and a secondary charge, but a sliding scale, instead of a fixed primary charge which prevailed in the old contracts, is provided.

TABLE I.—CHARGES FOR ELECTRIC CURRENT TO BE PAID TO THE COMMONWEALTH EDISON COMPANY BY CHICAGO ELECTRIC RAILWAYS UNDER PRESENT OPERATION BY SEPARATE COMPANIES.

Primary or Power Charge.

Maximum demand in Kw.—	Primary charge per Kw.
To and including 30,000	$1.25
Excess over 30,000 to and including 60,000	1.00
Excess over 60,000 to and including 90,000	0.917
Excess over 90,000 to and includ'g 120,000	0.875
Excess over 120,000	0.833

Secondary or Energy Charges.

Monthly energy consumption in Kw.-hrs.—	Charge per Kw.-hr.
To and including 5,000,000	0.400 cts.
Excess over 5,000,000 to and including 10,000,000	0.395 cts.
Excess over 10,000,000 to and including 15,000,000	0.390 cts.
Excess over 15,000,000 to and including 20,000,000	0.385 cts.
Excess over 20,000,000 to and including 25,000,000	0.380 cts.
Excess over 25,000,000 to and including 30,000,000	0.375 cts.
Excess over 30,000,000 to and including 35,000,000	0.370 cts.
Excess over 35,000,000 to and including 40,000,000	0.365 cts.
Excess over 40,000,000	0.360 cts.

Greater efficiency, to be obtained by (1) an improved load factor on the cables; (2) transmission and distribution economies; and (3) the joint use of substations by the railway and power companies, is expected to result from the avoidance of much duplication of transmission lines, cables and substations arranged for under the new contracts. The calculation of the maximum amount of current which the Commonwealth-Edison Co., the contractors, may be called on to deliver is based on the heaviest hourly morning and evening loads on any three consecutive days, so selected as to yield a total consumption of energy greater than would be obtained as the total of the corresponding six hours on any other three consecutive days. The power company must be prepared to furnish a maximum of 20,000 KW. or a demand equal to that established by the loads of previous months, if they exceed 20,000 KW. The additional power used to heat the cars in cold weather, when the air has a temperature below 15° F., is not taken into account in computing the maximum demand.

A correction to the prices given in Table I is provided, to be used when the cost of coal to the power company exceeds on the average $1.90 per ton delivered at the storage yards of the Edison company, as follows: If, during any two-year period, the price of coal having a heating value equivalent to that of coal from mines in Central Illinois exceeds on the average $1.90 per ton the consuming company is to pay an extra charge obtained by using the following formula:

$$\text{Total excess charge for coal} = \frac{2 \times KW\text{-hrs. (consumed in 2-yr. period)}}{2{,}000} \times (\text{Cost of coal} - \$1.90.)$$

This is equivalent to paying the average excess cost of coal at the rate of 2 lbs. of coal consumed per kilowatt hour.

The current will be metered in alternating current form at the high-tension terminals of the transformers where rotary converters are operated by the railways. Where the converters are operated by the power company the current will be metered at the delivery point to the direct current feeders. All power plants and substations of the elevated railway companies may be leased by the Commonwealth Edison Co. at rentals stated in the contracts so graduated as to cover the natural depreciation of the buildings and equipment.

A maximum scale of charges similar to those given in Table I are provided in the contracts which are to apply in case the railway systems are consolidated. These charges are given in Table II.

TABLE II.—CHARGES FOR ELECTRIC CURRENT TO BE PAID TO THE COMMONWEALTH EDISON COMPANY BY CHICAGO ELECTRIC RAILWAYS IF CONSOLIDATED

Primary or Power Charge.

Maximum demand in Kw.—	Primary charge per Kw.
To and including 75,000	$1.25
Excess over 75,000 to and includ'g 150,000	1.00
Excess over 150,000 to and includ'g 225,000	0.917
Excess over 225,000 to and includ'g 300,000	0.875
Excess over 300,000	0.833

Secondary or Energy Charges.

Monthly energy consumption in Kw.-hrs.—	Charge per Kw.-hr.
To and including 12,500,000	0.400 cts.
Excess over 12,500,000 to and including 25,000,000	0.395 cts.
Excess over 25,000,000 to and including 37,500,000	0.390 cts.
Excess over 37,500,000 to and including 50,000,000	0.385 cts.
Excess over 50,000,000 to and including 62,500,000	0.380 cts.
Excess over 62,500,000 to and including 75,000,000	0.375 cts.
Excess over 75,000,000 to and including 87,500,000	0.370 cts.
Excess over 87,500,000 to and including 100,000,000	0.365 cts.
Excess over 100,000,000	0.360 cts.

Annual Meeting American Society of Agricultural Engineers.—The American Society of Agricultural Engineers held its seventh annual meeting at the Great Northern Hotel, Chicago, Ill., December 29-31, 1913. Papers presented that were of interest to civil engineers were as follows: "Farm Sanitation, with Special Reference to Water Supply and Sewage Disposal," by Paul Hanson, Illinois State Water Survey; "The Design of Permanent Farm Buildings," by E. S. Fowler, Universal Portland Cement Co.; "Standardization of Wagon Construction," by E. Parsonage, John Deere Wagon Co.; "Concrete in Drainage and Irrigation," by C. W. Boynton, Universal Portland Cement Co.; "Extension Work in Agricultural Engineering," by F. M. White, University of Wisconsin; "The Five Winnipeg Motor Contests and Lessons to be Drawn from Them," by P. S. Rose, American Thresherman; "Suggestions Concerning a Motor Contest," by E. A. Johnson, International Harvester Co., and "Report of the Committee on Proposed Motor Contest in the United States," by L. W. Chase, University of Nebraska.

The following officers were elected for the coming year: President, W. F. McGregor, J. I. Case Threshing Machine Co., Racine, Wis.; first vice-president, J. L. Moury, Agricultural Engineering Department, University of Minnesota; second vice-president, W. T. Brandon, Avery Manufacturing Co., Peoria, Ill.; secretary and treasurer, F. M. White, Agricultural Department, University of Wisconsin.

Engineering and Contracting

**Devoted to the Economics of Civil Engineering Design
and to Methods and Cost of Construction**

Volume XLI. CHICAGO, ILL., JANUARY 21, 1914. Number 3.

Has the Last Word Been Said About Water-Bound Macadam?

Two articles suggest this query, the one by Prof. W. G. Fearnsides in our issue of Dec. 31, and the one by Mr. J. E. Tupper printed elsewhere in this issue.

Prof. Fearnsides advances the theory that macadam "binds" because of the force known as "surface tension" that exists in the water that fills the minute voids in the "binder." This theory was first announced in "Economics of Road Construction" (1901) by H. P. Gillette. While the theory is probably not perfect, it offers, nevertheless, an explanation of one of the causes of the binding action of clay, rock, screenings, fine sand, etc. To the "colloidal" character of certain soils and powdered rock may also be attributed some of the binding action that occurs in macadam. There may be other forces that assist the binding. The last word has not yet been said as to this interesting phenomenon which MacAdam himself so little understood.

Mr. Tupper now asks some new questions and revives a few old questions as to water-bound macadam. He questions, seriously, whether water-bound macadam is an obsolete form of pavement. He believes macadam to be preferable to concrete under certain conditions. He thinks that clay should be more frequently used as a binder. He believes that macadam should not be laid in a "trench." He sees no necessity of constructing macadam "like a filter bed." He finds no need of setting grade stakes by which to secure uniform thickness of macadam. Throughout his article Mr. Tupper is the scientific iconoclast. Apparently, however, he does not know that other engineers hold many of the opinions that he holds. Mr. Owen, for example, has used clay as a binder for macadam roads in New Jersey for many years, and so had other engineers before him.

Mr. Tupper makes an error that seems to need correction. Speaking of the cross-section for a macadam road recommended by Mr. Gillette in "Economics of Road Construction," Mr. Tupper says:

> Like all other roadmen, he [Gillette] is obsessed with the idea of the essential permeability of macadam, though he has pointed out in another place a method whereby it may be made practically impervious.

In attributing to Mr. Gillette a belief in the "essential permeability of macadam," Mr. Tupper is in error. Mr. Gillette does believe that water often gets under macadam and permeates the subsoil, without its having percolated through the macadam. This ordinarily happens in one or both of two ways. Where there is high ground adjacent to a macadam road, heavy rains or melting snows often saturate the ground to such an extent that water is forced, by the hydrostatic pressure of higher levels, up under the macadam; and this may happen in spite of drainage ditches. This is one of the reasons why macadam in "cuts" is so apt to be pounded out of shape by traffic in the spring of the year. Another reason for saturating of subgrades is the existence of snow and slush. These dam the ditches and the roadway, and cause the melted snow water to flow back under the macadam. This phenomenon is seen in its most objectionable form where the subgrade is rather impervious and where the macadam is laid in a "trench." These reasons, together with the fact that cheaper grading results, account for Mr. Gillette's abandonment of the old designs wherein macadam was laid in a "trench."

Mr. Tupper's belief in the economy of water-bound macadam under certain conditions is a belief that will not down. Bituminous macadam, concrete, brick, and other forms of road pavement unquestionably have their economic uses, and the field for each of them is rapidly extending. But there will always remain, we believe, economic justification for the dirt road, for the sand-clay road, for the gravel road, and for the water-bound macadam road, under certain conditions as to density and kind of traffic and unit costs of road construction. Engineers who have tended toward the complete abandonment of water-bound macadam will ultimately find that it has its economic uses. Mr. Tupper speaks a timely word.

Initiative—Applied and Misapplied.

We often hear it said that a certain person has—or lacks—initiative, this term being applied frequently to the young engineer. Take the case of the possible employer who has before him the young engineer's application for a job. Generally the first thing about which inquiry is made is character; the next, initiative. We do not believe that the question of initiative should be raised at this time; but that the employer more properly should seek to discover whether or not the young man is inclined to continue the studies begun at school and to familiarize himself with what has been accomplished. Experienced engineers in lecturing to engineering students sometimes deplore the fact—as they see it—that young engineers do not possess initiative. The result is that their statements are taken too seriously by these young men, who feel that they must assume a bold front and must appear capable of performing engineering tasks which experienced men would undertake only after serious investigation.

We do not believe that the young engineer can possess the proper kind of initiative until he has acquainted himself with the main facts relating to the problems at hand, which knowledge can be gained only by hard and conscientious work. After he has carefully studied what has been accomplished, then—and not until then—is he qualified to take a step in advance. Of course he may make a better "first showing" by immediately starting to work out an "inspiration" which in the end may develop into something of value, but only after repeated trials and a great waste of time and energy. The point which we wish to emphasize is that many of us are inclined to put the cart before the horse, instead of realizing that a careful study of the available data on the subject can profitably be made before the actual design is started, instead of waiting until experience teaches us that we are making very inefficient progress. The same tendency works to prevent a continuation of systematic study after graduation from college, with the result that our private libraries consist of only a few books—pocket size.

We have in mind a well-known engineer who has shown great originality in some of his designs, but we know that these designs were worked out only after an exhaustive study of the available data on somewhat similar structures. This man not only possesses a well-equipped library, but also has learned how to make use of it. His designs are not the product of inspiration, but rather of much preliminary study and investigation. His ability to reason and to draw logical conclusions is mainly the result of the knowledge and training gained by systematically studying that which others have accomplished.

We believe that the young engineer who cultivates the habit of study and investigation and who has respect for the ability and learning of those older and more experienced than himself will unconsciously acquire some of those qualities which tend to make him a leader. The first duty of the young engineer is to make his services of value to his employer, and he can best do this by first acquiring a reputation for reliability and hard, conscientious work.

Queries Directed to Engineering Societies.

Within the present month there have been or will be held the annual meetings of perhaps a score of engineering societies. The year of 1913 has opened wider four ways for the advance of the engineer. These ways are management engineering, valuation of public utilities, city management, and public service commissionership. What are engineering societies doing to put before the public the special claim of the engineer for useful official participation in these activities?

Engineering societies know that the majority of the tasks involved in public utilities valuation and regulation and city management are engineering tasks, but the public, at whose will these activities have been instituted, does not know. It has to be told and its favorable opinion influenced. Who, if the engineering societies do not, will give the information required and exert the needed influence?

Engineers are engaged in valuation work, in public service management and regulation, in city public works, successfully as servants under lawyer and financier and politician direction. In a number of instances an engineer is successfully exercising directorship What in any community has the engineering society done toward placing engineers in the membership of public service commissions, in executive charge of valuation work, in the office of city manager?

Engineering societies have materially aided the progress of the engineer so far as such accomplishment can aid it by working to perfect his technical attainments. This is a fundmental service, but it is only a part of the service that the engineer requires to secure the heritage of public appreciation to which he is entitled. From this viewpoint what is the efficiency of the service of the engineering society?

An Epitome on Municipal Engineering

The letter quoted herewith expresses, with much felicity, the requirements and limitations commonly exacted of and imposed upon municipal engineers by city officials. This letter was received by a Kansas City engineering firm from the mayor of an Iowa city. In a previous letter the mayor cautioned the engineers regarding the necessity of strict economy in the design of the contemplated improvement, and they replied assuring him that all considerations of economy would be given due weight consistent with good construction requirements. The reply of the mayor follows:

> I did not mean to convey the idea that we wanted an inferior or poorly constructed reservoir. On the contrary, I want no stone left unturned to insure a perfect job. I want a reservoir that will stand an earthquake and never quiver, one that can be taken out bodily and carried off, one in which there will never be a crack, one that will never leak a drop, one that will stand there for ever and ever, amen.
>
> What I want to avoid is the spending of money. I want the above described reservoir for little or nothing. I think I make myself clear.

Edward Hervey Lee, President Western Society of Engineers.

Edward Hervey Lee was elected President of the Western Society of Engineers on Jan. 7, 1914. Mr. Lee has been since 1898 chief engineer of the Chicago & Western Indiana R. R. and the Belt Railway of Chicago, two railways which provide trackage facilities for a number of important railways entering Chicago. He began his railway work as rodman for the Scioto Valley R. R. in 1880 and, with the exception of about four years in private engineering practice and engineering contracting, he has been in railway engineering service for 33 years. Mr. Lee's enginering work, like that of all railway engineers who have started as a minor unit in a survey corps and have advanced by steps to chief engineership, has included all kinds, but since 1898 it has been exceptional in the amount of city grade elimination and terminal yard work for heavy traffic.

Edward H. Lee was born Jan. 29, 1863, at Dayton, O. His schooling was received at Ohio University, Athens, O., and at Wooster University, Wooster, O. He began engineering work in 1880, when about 17 years old, as a rodman on the Scioto Valley R. R. This railway, now a division of the Norfolk & Western Ry., was then being located and constructed between Portsmouth, O., and Ashland, Ky. The young engineer's next engagement took him to the opposite end of the state where at Cleveland he drove the first stake in the location survey east of Cleveland for the New York, Chicago & St. Louis R. R., commonly known as the "Nickel Plate." During the construction of this railway Mr. Lee was connected with engineering corps working both east and west of Cleveland and was stationed at Fort Wayne, Ind., when the road was opened for traffic from Buffalo to Chicago. His work on the Nickel Plate raised Mr. Lee to responsible rank in railway engineering and the record of his future work need only mention that of special character.

As assistant engineer for the Ohio River R. R., now the Ohio River Division of the Baltimore & Ohio R. R., Mr. Lee had charge of the heavy construction work along the rock cliffs and hillsides that form the West Virginia bank of the Ohio River from Wheeling to Parkersburg. Mr. Lee then held positions twice with the Wisconsin Central and once with the Union Pacific. His first engagement with the Wisconsin Central was as resident engineer on the construction of the main line into Chicago. He then became assistant engineer for the Union Pacific on the construction of the branch lines of that road in Nebraska and Kansas. Returning to the Wisconsin Central Mr. Lee was connected with the construction of the first large ore dock built at Ashland, Wis.

In 1887 Mr. Lee was appointed office engineer of the Elgin, Joliet & Eastern R. R., and in that position he located the branch line that was constructed to the coal fields. Later he was promoted to be chief engineer and among other tasks he built locomotive shops at Joliet and located and constructed extensions from McCord to Porter, Ind., and into the cities of Joliet and Aurora, Ill. Mr. Lee now turned from railway work and from 1893 to 1897 was employed on the construction engineering force of the Sanitary District of Chicago, which was then building the Chicago Drainage Canal, and also in private engineering practice and in contracting.

As engineer with the Sanitary District Mr. Lee had charge of the field work for the dredging operations for Sections A and B of the channel—the two earth sections west from Bridgeport where the canal connected with the Chicago River. He also had charge for the district of the dredging of the Chicago River, and the construction of the Western Boulevard Bridge over the main canal. His private practice during this period, 1893-7, included the construction of three miles of 8 and 10-ft. sewers in Chicago, the construction of some 50,000 cu. yds. of concrete retaining wall along the rock sections of the western end of the Drainage Canal, and the construction of a diversion channel for the Michigamme River, in the upper peninsula of Michigan.

In 1898 Mr. Lee returned to railway work, being first principal assistant engineer in charge of grade separation at 16th St., Chicago, and later appointed chief engineer of the Chicago & Western Indiana R. R. and the Belt Railway of Chicago, the position which he now holds. Mr. Lee's work now became, because of the position of the roads named as terminal and transfer lines for several large systems, very largely of three classes: grade crossing elimination, terminal yard construction and bridge construction.

Grade crossing elimination as prescribed by the Chicago ordinances is practically complete track elevation; the prevailing construction being embankment fill held by retaining walls for the portions between streets and over crossing streets being steel or concrete solid floor viaducts or bridges. The programme for grade elimination on the Chicago & Western Indiana and the Belt Line roads included some 22 miles of line. With the exception of one crossing of the Belt Line and the Chicago Burlington & Quincy R. R., built in 1891, all this 22 miles of track elevation has been under the direction of Mr. Lee; some 15 miles of the work have been completed and the remainder is in process of construction. The work so far done has involved some 250,000 cu. yds. of concrete wall, abutment and foundation work; some 20,000 tons of structural steel work and some 5,200,000 cu. yds. of sand and earth embankment between retaining walls.

Besides the bridge work involved in track elevation proper Mr. Lee has directed the engineering and construction work of a number of bridges over stream crossings and two of these are of exceptional character. One is the 1,000-ton Strauss bascule of 186 ft. span, double-track, built in 1909 over the Grand Calumet River. The second is the notable four-track swing bridge, built in 1900, over the Chicago Drainage Canal. The superstructure of this bridge weighed 1,540 tons and the moving portion weighed 1,480 tons; this bridge cost nearly $182,000.

In building and yard work, the 47th St., 55th St., 68th St., and Englewood stations; a 51-stall roundhouse, powerhouse, machine shop, storehouse, and coal, cinder and inspection pits at the 49th St. engine terminals; a supply house for the Chicago & Western Indiana R. R., a thawing out house and a Pullman supply house at the 51st St. yards, have been constructed under Mr. Lee's direction. On the Chicago & Western Indiana R. R. he has constructed yards for the Chicago & Eastern Illinois R. R. at 35th St., for the Wabash R. R. at 47th St., and for the Erie R. R. at 51st St. The joint yards at 51st St. have been extended and improved, coach yards constructed and at 65th St. the Englewood team yards have been built. On the Belt Line the West Chicago yards, the West 22nd St. yards, the Rockwell St. yards and the South Chicago yards have, under Mr. Lee's direction, been enlarged and improved. A new clearing yard is now under construction.

Mr. Lee became a member of the Western Society of Engineers Nov. 7, 1889. In 1899 he joined the American Railway Engineering Association as a charter member, and in 1900, the American Society of Civil Engineers. He is also a member of the Engineers' Club of Chicago. In the American Railway Engineering Association Mr. Lee has been a member of the Committe on Track for one year, of the Committee on General Contract Forms for two years and of the Committee on Arrangements for three years. In his engineering society work, Mr. Lee has been noted particularly for his labors on committees and in the advance in numerous similar ways of the technical work of the society, rather than for the frequent presentation of engineering papers. His genial and kindly consideration of his associates is the characteristic by which he is perhaps best known aside from his professional ability.

General

The Engineer-Manager Plan of Municipal Government as Successfully Applied at Abilene, Kans.

In an editorial published in our issue of December 31, 1913, we commented upon the growing popularity of the city manager plan of municipal government. The engineer-manager plan was held to be especially advisable for towns too small to employ regularly a city engineer. The experience of the city of Abilene, Kan., was cited as a successful application of the plan in a city of 5,000 population. The present article gives further discussion on the general features of the plan and gives data drawn from a trial of the plan at Abilene during the past eight months. The data and discussion are from a paper by Mr. Kenyon Riddle, engineer-manager of Abilene, before the Kansas Engineering Society.

The success of any good business is possible if it is controlled properly. Imagine some piece of construction work being carried on by a body of men and each man having equal authority, or a railroad under such conditions. Nevertheless, this is the encumbrance which a great deal of city work struggles against. However, work done under standard specifications is saved by the clause, "All instructions shall be given to the contractor through the engineer." This clause guards against the unlimited confusion that would result if no one person were made the head.

In maintenance work and the general business of the city, though, is where confusion arises from too many men giving directions to the city's employes and from giving individual orders. Every important move of the commission should be acted upon by the body as a whole, then delivered to one man for disposal or distribution to the proper heads or departments. This removes individual action by any one member of the board, and it also cancels any feelings of jealousy among the board members. It makes the commission as a body responsible for the act and the one man is made to account for putting this act into service. The manager, being a salaried man with no definite term of office, naturally would strive to give his best service, since the length of his term depends upon his capability.

Preferably the manager should be without extensive interests in the town or city which he serves. This makes it more possible for him to give impartial views in matters concerning employment of labor and purchasing of material. His preparation should consist in a thorough education along the lines of general engineering rather than a knowledge of a few special branches. This will make it possible for him to advise and check up, in a general way, every act of the engineering department. He should also have an understanding of values and be able to estimate all classes of construction and maintenance work, be economical and able to establish systems of record-keeping and cost-keeping, and have a general conception of business methods and finances. In addition to this he should have an understanding of the city's sanitary and social conditions, which exert an important bearing upon public welfare. Inefficiency here causes every imaginable trouble. Here is involved the street cleaning and disposal, water system and supply, sanitary sewer system, drainage system, the working and living quarters of every class of people, the transportation facilities, and numerous other items.

A general knowledge of all of the above particulars is not too much to expect of a city manager. However, since men have not prepared themselves for this particular situation it may be well to try the new system of control in smaller towns, for in these places some of the qualifications of manager can be stricken out entirely and others are of much less importance. On the other hand, in the larger cities the different departments are more complete in themselves and would require less direct supervision and advice from the manager.

Since these requirements have been enumerated it is not at all difficult to see that the great majority of the city's affairs are of an engineering nature and should be managed by an engineer.

Since the engineer-manager plan is especially applicable to the average sized towns of 3,000 to 25,000 or 50,000, and since its success is more probable in these places, let us confine ourselves to this particular application.

In order to demonstrate the importance of the adoption of this more simplified form some of the defects of the older and more complex forms might be brought out. These innumerable defects have been the cause of an awakening on the part of the people who wish better administration.

People have commenced to see that public affairs as a rule have not been handled in in a business way, but rather along lines of politics, and when business becomes mixed with politics then efficiency is replaced by inefficiency. Instead of each member of the governing board exerting his efforts along the lines he believes to be best and in a manner desired by the people, he is very often misled by reason of political views. Appointments are very often made because of favoritism or by reason of political debt. Also due to politics, every citizen is not treated on an equal basis. Improvements are centered around certain localities, and are not distributed in an impartial way. Everyone is aware of the fact that money is wasted in nearly all public business. Some of the dissipation of tax money is often due to graft, etc., but most of it is the result of poor management, lack of harmony among the board members and a lack of knowledge along particular lines, especially in the engineering department of smaller towns and the purchasing department of nearly all municipalities. No doubt the larger cities lose money in their engineering departments, due to lack of management, and the department is possibly hampered by politics. Nevertheless they do not lack in principles of engineering practice as do the smaller towns.

Therefore, in consideration of the small town the main object in order to secure efficiency where it is most lacking is to devise some scheme whereby the town can feel justified in having the services of an engineer at all times, and this remedy lies in the engineer-manager plan. It will be shown later on how the creation of this office is possible for any town and how it can be made remunerative.

We might account for wastes due to a complex system of management. The particular duties of different officials and departments are not well defined, thereby causing confusion and resulting in loss of time, which is loss of money.

Wastes due to lack of harmony are quite common. This lack of harmony is usually traced back to differences of politics or possibly jealousy among the different members. Without harmony there is no unity and without unity all advancement is retarded.

But the principal dissipation of funds comes from lack of knowledge, and this particular shortage is most often conspicuous in the engineering department, and since the great majority of the city's business is of an engineering nature this makes up the greatest waste. This particular argument best fits a small town, since these are the towns which are believed to be too small to feel the necessity of employing an engineer the year around. These are the places where a lack of knowledge makes up the principal reason for receiving about three-fourths value on each dollar spent in taxes. The permanent improvements and systems in such places are the nucleus for possible larger systems. Therefore they should be carefully planned so as to meet future conditions. They should be accurately recorded also.

If a town or city had some means of figuring the money wasted in prospecting for lost underground systems, in the improvements buried and never found, in systems that are built with no idea of future growth and that have to be abandoned years before they are worn out, in the grading and re-grading of streets, in sinking hundreds of dollars into permanent culverts and waterways built with no idea of future drainage and that have to be broken out when pavement is laid, it would be no difficult matter to prove the economy of city planning and of proper management.

How could anything else be expected from a council or commission composed of bankers, lawyers, farmers and men who have devoted their efforts to their professions and who are in active business and cannot neglect it to make a study of engineering so as to carry out some drainage proposition or such for the city, their salaries being anything from $1 to $25 per month?

As soon as a town reaches a size which justifies permanent improvements, from that time on systematic methods should be carried out, and a capable man who is paid a fair salary and feels permanency in his position is more apt to be able to do these things than a council or commission. It very seldom is the case that the successor will follow plans laid down by his predecessor. On the contrary, not only will he refuse to follow his predecessor's plans, but he will exert his efforts in tearing them down.

With an engineer-manager it is possible to get the very best and most active business men to serve on the commission, since their duties are reduced to a few hours a week. Furthermore, it is generally admitted that the holding of a political job is not a successful means of increasing one's business, but under the new plan all complaints and grievances may be put to the manager, such as come up every day of a lack of water pressure, sluggish sewers, poor drainage, etc.; and since he is not elected by the people, but appointed by the commissioners, he may be free to act as one whose best judgment is not overpowered by some political fear. By hiring a manager the hindrances of politics are eliminated and *professionalism* is substituted.

Can anyone argue that this will not bring about efficiency? The average person cannot realize that paying what now seems like an exorbitant salary for expert men is economy. The principal reason is that one *can* see the expense of salaries, but can *not* discern the huge waste due to poor and inadequate management. This waste invariably exceeds the expenditures that might be made for men capable of handling public affairs.

Now, the way to eliminate these objectionable features of the old style of administration is to adopt commission form of government as the executive board and as representatives of the people, and for them to select, by purely business methods, an expert through whom they can look for professional advice. As a result of this professional advice many blunders in the town's affairs can be overcome.

Every person here who has had any experience with the different systems of a city well knows the deplorable condition which some of these are in. The distribution of the water comprises a length of misfit, complicated pipes and valves, with absolutely no records of the underground system—the result of having untrained, inexperienced men who wish to experiment along the lines of engineering, all at the expense of the city. Due to the fact that the town had no engineer, they thought it would be economy to plan and supervise their own systems and extensions. During the time that the town is small and when these errors are being made is the very time to put into practice every method of city planning, since this is the foundation of a possible larger system. And, by all means, permanent records should be kept of all underground work.

It is needless to say that the sewer systems suffer the same inexcusable poor treatment. These mistakes all bring about complications and unnecessary expense to the city.

Also think of the money wasted in drainage systems. This is one feature on which the incompetent street commissioner of a small town professes profound knowledge. Costly and permanent waterways, etc., are built to care for drainage, with absolutely no idea of drainage, of adequate capacity, of the future outcome or development of the scheme. In a few years, the street is paved and it then becomes necessary to tear out hundreds of dollars' worth of these permanent improvements. The only means of overcoming these items of waste is for the city to proceed along lines which will insure expertness. If they were to employ an outside engineer for these less important things doubtless it would be too expensive, but the proper procedure is to devise some method whereby the town will have an engineer at their service at all times. Salvation lies in the adoption of the engineer-manager plan.

Now, to make this newly created office remunerative: The employment of an engineer as manager no doubt will be considered too expensive, especially for a small town. The thoroughness of record keeping which it is possible to maintain would more than obviate this objection. But there are several ways in which an income to the city's general fund can be realized by the newly created office: No matter how small the town, all sewer work, plumbing, wiring, water mains and services, meters, etc., should be inspected, and the small cost for doing this should be charged to the property owner. Then there will be the ordinary expenses of surveying lot lines, giving grades for buildings and sidewalks; this would be charged to the property owner and turned into the general fund. Also, there will be the usual engineering for sewer and water extensions, curbs and pavements. And finally, if the manager has charge of the street work, the street commissioner's salary may be added. The aggregate of these different amounts, if the town were doing its share of improvement work, would average from $100 to $150 per month, a sufficient amount to employ a competent engineer, without extra draft on the city's funds.

To prove that this is not all theory let us consider the savings already made in Abilene, Kansas, a town of 5,000 population, which town has been controlled by commission form of government and an engineer-manager for the past eight months. During this time the following improvements have been made:

Street improvements	$80,000
Ornamental street light system, consisting of five-light and single-light concrete standards, cost of installment, city's share	1,200
Light company's share	6,800
(The city has a contract with the light company to pay $3,000 a year for three hundred 10-watt lamps; about one-half of these are burned every night in the week, and the balance are burned about one-half the time.)	
Water works improvements, consisting of general overhauling and new pump at plant and booster pump	5,275
Water main extensions	2,910
Sewer extensions	2,136
Making a total of	$98,321

Later on there will be shown a few examples of savings as a result of proper management. Credit for these savings is given to the methods employed, such as: Preliminary estimates which save the city from overcharges; plans and specifications, which insure better workmanship and speed; system; supervision and inspection, and a final measurement and estimate, which insure the city of fair settlement.

A summary of the several small sewer and water extensions exhibits the following marked differences. This takes into account work done by the old administration and that done by the commission body with a superintendent or manager. Somewhat different methods have been employed in the recent work. The city has purchased all of the material and let the labor to the lowest responsible bidder. All other conditions are practically the same. That is, cost of labor and material and all other features, such as depth of trenches, quality of soil, etc., are figured the same. The comparison is based on trenches averaging 8 ft. in depth, number one 6-inch vitrified sewer pipe, flushtanks and manholes built under the same identical circumstances.

Past methods show the following:

Excavating and refilling trenches averaging 8 ft. in depth	40.8 cts. ft.
Furnishing and laying 6-in. vitrified sewer pipe	15.5 cts. ft.
Furnishing material and building flushtanks	$66.00 each
Furnishing material and building manholes	$40.80 each

New methods:

Excavating and refilling trenches averaging 8 ft. in depth		25 cts. ft.
Laying 6-in. vitrified pipe	5 cts. ft.	
Furnishing on works 6-in. vitrified pipe	10 cts. ft.	
		15 cts. ft.
Labor on flushtanks	$10.00	
Material in flushtanks	41.80	
		$51.80 each
Labor on manholes	$ 8.00	
Material in manholes	15.00	
		23.00 each

The saving is—in consideration of the amount done this year:

	Difference each item.
4,721 lin. ft. of excavation and refilling	$ 745.92
4,721 lin. ft. of furnishing and laying	212.44
3 flushtanks	42.60
4 manholes	71.20
Total amount saved	$1,072.16

on this one very small item of the city's business during the eight months of city management.

To save enumerating the total savings in the waterworks department I have figured that the difference between the unit costs under the old and new systems represents 8 cts. per lineal foot and for the 5,010.0 ft. laid this year the saving would be $400.80.

The cost of cleaning the pavement and disposing was cut from an average of 40 cts. per 1,000 sq. yds. to 36.2 cts. per 1,000 sq. yds. Therefore on the total area of 2,215,797 sq. yds. covered in order to keep 92,000 sq. yds. clean for six months there was a saving of $842.00.

It would be too monotonous if I were to itemize every saving that has been accomplished by the more simplified form. However, in every department there has been a saving. The saving on the street cleaning was doubtless due to system. At present there is a report turned in by the street commissioner, showing the streets cleaned and the cost. From this report the number of sq. yds. is figured and the cost per 1,000 obtained. From this data different methods of cleaning have been tried until the most satisfactory method is obtained.

The dragging of dirt roads used to be considered too expensive and was therefore omitted. No one had any idea how many miles of dirt streets the town had which could be considered as arteries of traffic nor how much a mile the dragging should cost. In order to prove that all of the more important streets could be dragged at a nominal cost the streets were gone over and two districts were laid out. For each of these districts a log-sheet was made, showing the streets to be covered, the number of times to cover each of them and the number of miles traveled. One district has 16½ miles to cover and the other contains 19 miles; when the streets need dragging teams are employed and each of the two outfits is paid $6 for following out the course as listed on his log. During the past eight months—of course some of these months were quite dry—the dragging has cost $51.

Now besides the $2,315 which has been saved, as itemized here, the city has a complete set of records of all underground work and of every survey made.

In order that all important work shall be authorized by the commissioners as a body, with a clear understanding on their part, a report of the coming week's work is made up, with a preliminary estimate of the cost and an account of the benefits derived. This is presented by the manager at the meeting of the board and after the work is discussed the commissioners order that which they deem necessary. The manager then takes charge and at the next meeting of the board he reports on its progress; or if it is completed he submits the final estimate. With this procedure the commissioners are thoroughly acquainted with all of the city's affairs.

With all of the principles which have been discussed here and with the conclusive evidence of efficiency which has been cited as attained by the businesslike methods which are possible in the engineer-manager plan there seems to be absolutely no question that it is the ideal form of public control, particularly for municipalities, and probably most applicable to the smaller towns while in its experimental state.

The Economy of the Electric Motor Drive for Contractor's Hoists.

Contributed by W. H. Easton, Westinghouse Department of Publicity, Pittsburgh, Pa.

The question, "Is the electric motor driven hoist cheaper to operate than the steam hoist?" is frequently asked. The electric hoist has some obvious advantages over the steam hoist, making it more desirable to use if current is available, provided the cost of operation is not out of all proportion to the benefits received. But the question is not an easy one to answer in dollars and cents. That is, it is very difficult to compile figures that would show for example that a steam hoist completed a job for $100 and an electric hoist did the same work for $125, or perhaps for $80; because the cost of operating a hoist is composed of a great many factors, many of which are difficult to estimate, while others vary widely with varying conditions. It is, however, practicable to compare the conditions under which the two types of hoists operate and to determine whether under a given set of conditions the electric hoist would or would not be more economical than the steam hoist. If this is done it is believed that the electric hoist will be found the cheaper for average work where electricity is available for the following reasons:

The electric hoist does not require a licensed engineer. Almost any intelligent workman can learn to operate it, because it has no boiler, the control is very simple, and the motor requires no attention when operating. A single handle controls all motor operations. Throwing it one way starts the hoist in one direction, the speed depending upon how far the handle is moved; throwing the handle in the opposite direction reverses the hoist, the same speed range being also available.

Having no boiler, the electric hoist requires no fireman, no fuel or water. There are no ashes to handle, no frequently objectionable smoke and exhaust, and no sparks, which eliminates the fire risk. The electric hoist is also lighter and more compact than the steam hoist with its boiler, and is therefore cheaper to move.

Where the steam hoist is driven from an independent boiler, steam pipes, which are leaky and awkward to handle, must be laid. The electric hoist, on the other hand, gets its power from flexible cables which can be run anywhere with ease.

Another advantage of the electric hoist is that it consumes power only when actually in use, and when at rest involves no expense for power whatever, whereas with a steam hoist, steam must be kept up, and frequently the stand-by expense exceeds the actual operating expense.

The motor-driven hoist is in general less likely to be out of commission than a steam hoist. It can be started at any time without delays for steaming up. There is nothing about it to freeze up and hence it is independent of weather conditions. The simple construction of the motor, with its two bearings only and no reciprocating parts, insures minimum delays for repairs, and as for reliability, motors designed especially for crane service are as strong and rugged as any steam engine.

There remains nothing but a consideration of the cost for current as compared with that for coal. This will naturally vary with local conditions, but it can be stated that in general the cost of operating an electric hoist will not be greater, all factors considered, than that of operating a steam hoist.

For example, with a coal hoist in Pittsburgh, where a motor was directly substituted for a steam engine, all other factors remaining the same, the following results were obtained:

Cost of coal per month..........	$ 60	
Cost of water per month.........	15	
Cost of electric power per month		$ 77
Wages of engineer per month....	125	
Wages of motor operator........		75
Totals	$200	$152

Thus the electric hoist showed a saving of $48 per month. But it also proved itself able to handle more coal. With the steam hoist, a bucket containing 42 bushels was lifted every 60 seconds, whereas the electric hoist required only 50 seconds for the trip, because it could be accelerated more rapidly. Hence in a 10-hour day the electric hoist can perform 120 more trips, or handle over 5,000 bushels more than the steam hoist.

Sewerage

The Design and Construction of a Plant for Complete Treatment of Sewage, Chester County Home, Pennsylvania.

Contributed by P. E. Mebus, C. E., and F. R. Berlin, C. E., Philadelphia, Pa.

In recent years, sewage treatment plants have been built at nearly all of the state and county institutions in Pennsylvania. The most recent of these is the plant at the Chester County Home, where the insane and indigent of the county of Chester are provided for. This institution, with a population of nearly 600, including attendants, is situated about six miles west of West Chester, on high ground overlooking the valley of the Brandywine Creek. The buildings consist of the Home for the Indigent, the Insane Hospital, Nurses' Home and the farm buildings.

In order to determine the capacity of the new plant, measurements were made which indicated that the total sewage flow for a 24-hour period would be about 42,000 gals. The average rate of flow between 7 a. m. and 7 p. m. was found to be about 2,000 gals. per hour, while the maximum rate was found to be nearly 3,000 gals. per hour.

The present plant was designed to treat the sewage flow of 1,000 persons, which at 70 gals. per capita would amount to 70,000 gals. per 24 hours. Provision has been made for additions to this plant, which will double its present capacity so that ultimately it will be able to treat 150,000 gals. of sewage every 24 hours.

The sewers are built on the separate system and only the domestic sewage and laundry wastes reach the treatment plant. The storm water is collected and disposed of at convenient points.

PAST HISTORY OF SEWAGE TREATMENT AT THIS INSTITUTION.

About 12 years ago, an irrigation field, having an area of about ½ an acre, was laid out, and a syphon was installed for dosing it. Unsuitable soil and improper distribution of the sewage over the field caused it to become a nuisance and a menace to health. The soil became what has been called "sewage sick," and oxidation was no longer possible. The continuous flow of sewage scoured the surface and formed artificial channels, through which it ran into the Brandywine practically in its crude state.

The Pennsylvania State Board of Health ordered the directors of the Home to install a modern plant and abandon the older method entirely. Plans were prepared, approved by the state officials and construction work began shortly thereafter. The present plant involves no new principles, although it has been designed and built to secure an effluent of exceptionally high degree of purification, which is necessary to protect certain water intakes situated farther down the stream. Local conditions also demanded a prevention of nuisance. It occupies a site near the old irrigation field, and about 1,000 ft. south of the institutional buildings.

NEW TREATMENT PLANT.

The thoroughness of treatment is indicated by the units of which the plant consists, viz., one primary settling tank of the Imhoff type; percolating filter and dosing chamber; hypochlorite disinfection house; secondary settling tank; and sand beds for drying sludge. A plan of the works exclusive of the sludge drying beds, is given in Fig. 1. A longitudinal section through the plant shows the chief elements in their relative elevations, see Fig. 2. The necessity for simplicity and facility of operation was fully appreciated in designing this plant. With occasional instructions from the authorities, an inmate of ordinary intelligence is able to operate the plant with a medium amount of attention and yet secure an effluent of the necessary degree of purification. A general view of the plant is shown in Fig. 3.

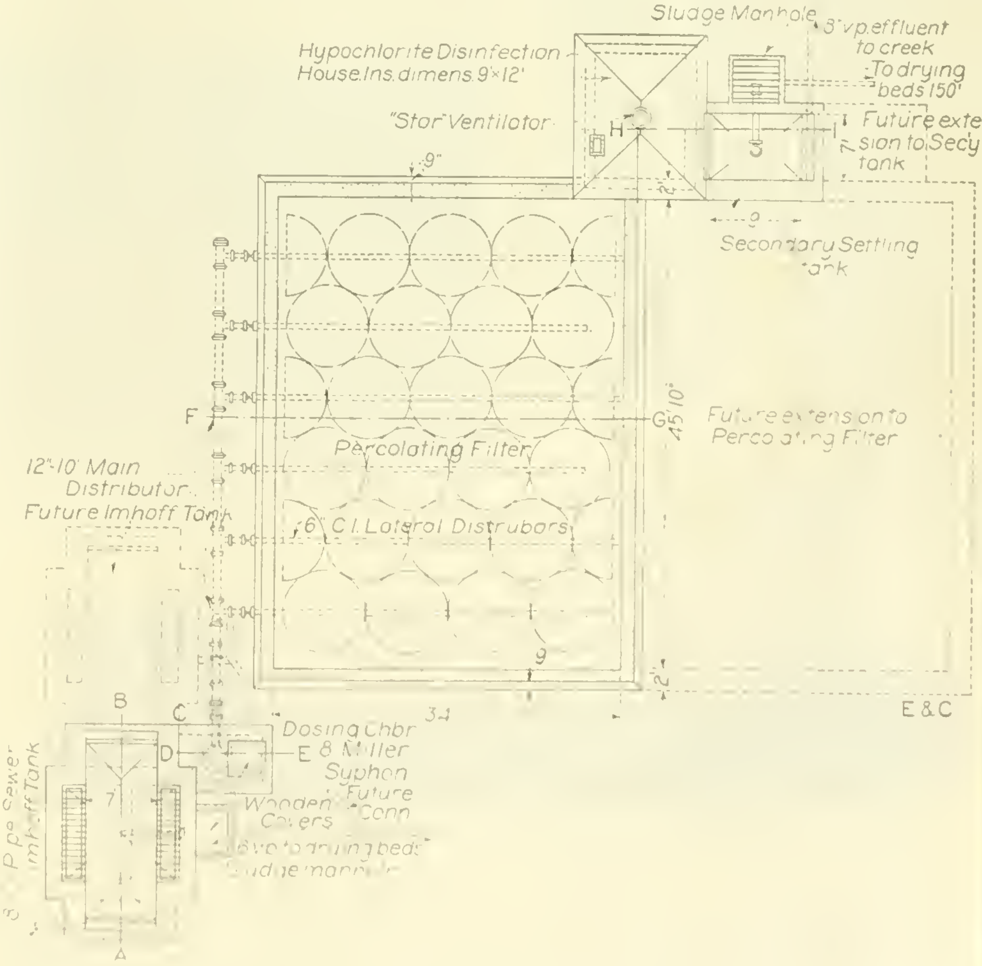


Fig. 1. Plan of New Sewage Treatment Works at Chester County Home, Pa. (Sludge Beds Not Shown).

Imhoff Tank. The Imhoff tank was one of the first in this country to be designed along Dr. Imhoff's ideas for a rectangular tank. It has practically a square sludge decomposing room with a hopper bottom, while the upper or settling chamber is made rectangular with overhanging ends, to provide an adequate detention period for the dilute sewage. The two compartments are connected by 6-in. slots. The settling chamber has normally a three-hour detention period, while the sludge decomposing room has a storage capacity for about four months. It was not deemed necessary to increase the latter because of the climate and the sheltered location of the plant. The sludge is removed in the usual manner, i. e., by a differential head of about 9 ft. The 6-in. cast iron outlet pipe extends nearly to the apex of the hopper bottom and the flow therefrom is controlled by a 6-in. gate valve located in a manhole adjacent to the tank. From the manhole it flows to the drying beds by gravity.

The sewage is distributed and collected by weirs extending the full width of the settling chamber, which insure a nearly uniform distribution. The bulk of the solids in the crude sewage are removed in this tank and settle into the decomposing room below.

The construction of the tank, with the exception of the diaphragm walls, is unique, in that it was built entirely of stone masonry laid in 1:2 cement mortar. The Home has a quarry on its property where an excellent grade of limestone was available at almos no cost. Masons for laying the walls could also be secured near at hand. A comparison of the cost of masonry wall built under these conditions with a concrete wall requiring expensive formwork, proved beyond a doubt the economy of the former type of construction. The inside faces of the walls were left rough and afterward plastered with cement mortar and given a smooth finish. The diaphragm walls were built of 1:1½:3 reinforced concrete and were carefully joined into the stone masonry. A pleasing and finished effect has been obtained by placing a concrete coping along the tops of the walls.

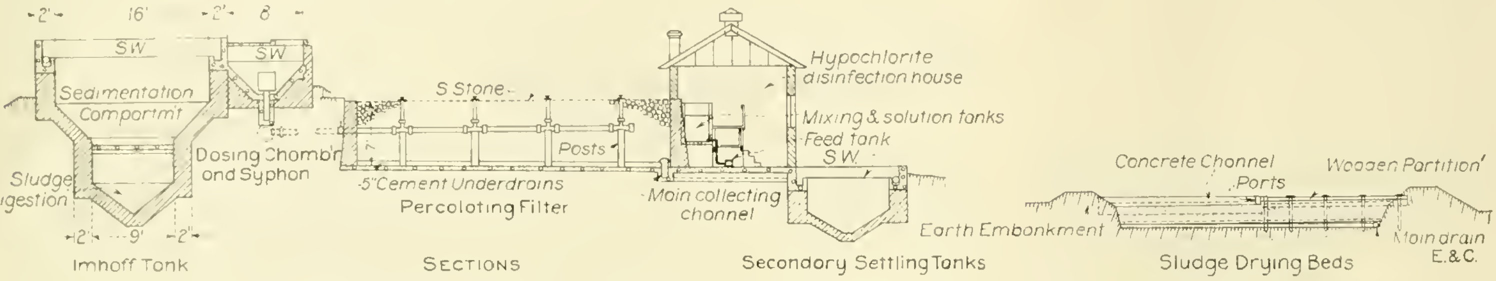


Fig. 2. Section A-B-C-D-E-F-G-H-I of Fig. 1 and Transverse Section of Sludge Drying Beds.

The mean velocity in the sedimentation chamber has been calculated as being practically 0.5 m.m. per second and the maximum velocity at the surface of the sewage will not exceed 1 m.m. per second. The velocity at the slots must necessarily be as near zero as possible.

Dosing Chamber.—The dosing chamber is located immediately adjacent to the Imhoff tank, and connected thereto by a continuation of the effluent channel. Before entering the chamber, the Imhoff effluent is passed through a ⅜-in. mesh wire screen to retain foreign substances which would have a tendency to clog the nozzles. The dose of about 350 gals. is discharged into the distributing piping by an 8-in. Miller syphon drawing 3 ft. of water. All of the feed pipe and the specials are cast iron water pipe, American Water Works Association standard for 100 ft. head. The joints were made of lead poured hot and afterward caulked.

Percolating Filter.—The superficial area of the percolating filter is nearly .04 of an acre. The average rate of filtration will be approximately 1,400,000 gals. per acre per day, and the maximum rate will not exceed 2,000,000 gals. per acre per day.

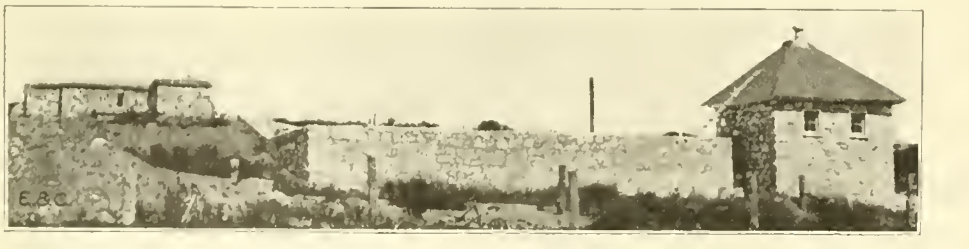


Fig. 3. General View of Sewage Treatment Plant at Chester County Home.

The sewage after leaving the dosing chamber flows in turn through the main distributing piping, the 6-in. lateral distributors, and the 3-in. riser pipes, from which it is sprayed on the stone by 27 Taylor hexagonal type nozzies of which 6 are used for half sprays. The lateral distributors are laid in the body of the filtering material and are supported by concrete posts running to the floor. The filter media, which is 7 ft. deep, is local limestone nearly white in color, the sizes ranging from 1½ to 4 ins The floor on which the cement underdrains are laid, slopes toward the right end along which is built the main collecting channel, which carries the effluent to the disinfection house. A view of the filter in operation is shown in Fig. 4.

Hypochlorite Disinfection House. — The hypochlorite house, which is built of cut stone masonry laid in cement mortar, is 9 ft. by 12 ft. inside dimensions. The main effluent conduit is laid under the floor and has obstructions placed therein to thoroughly mix the filter effluent with the hypochlorite of lime solution. The building contains the reinforced concrete mixing and solution tanks, and a float controlled constant head tank for adding the solution at uniform rates. The chloride of lime is mixed by hand and in sufficient quantity to last about 30 hours. This solution is allowed to run into the percolating filter effluent at uniform rates during both day and night, although the night rate is decreased to correspond with the decreased sewage flow during that time. The rate of adding and the strength of the solution are so adjusted that available chlorine is added at the rate of about 2½ parts per million.

Secondary Settling Tank.—The disinfected effluent flows through the main conduit and into the secondary settling tank, which is rectangular in plan and has a hopper bottom. The filter effluent is distributed and collected by weirs as in the Imhoff tank. The sludge which flows to the drying beds by gravity is removed by a differential head, similar to the scheme in use at the Imhoff tank. The detention period in this secondary tank has been made one hour as the designer's experience along this line has shown that this time is sufficient. The effluent is discharged into a small brook which carries it to the Brandywine. No nuisance is created in this brook due to the high degree of purification obtained in this plant, although the dilution ratio ordinarily will not exceed 1:5.

Sludge Drying Beds.—The drying beds have an area of .023 acres, which is rather excessive for sludge drying purposes only, in a plant of this size. It is, however, contemplated to use a part of this bed occasionally as a sand filter and for this reason a larger area was utilized than was really necessary merely for sludge drying. The plant piping is arranged accordingly. The distributing trough which is carried through the middle of the bed on a masonry wall, has four outlets, i. e., two for each bed, all of which are equipped with stop planks. The stone masonry wall besides supporting the trough acts as a dividing wall, so that one side of the bed may be used independently of the other. The media consists of 21 ins. of local sand spread over graded gravel, in which the underdrains are laid. The embankments surrounding the filters were made of clean earth sodded both inside and outside. The drain water is conducted to an old dry room through which it flows to the above mentioned Brandywine Creek.

CONSTRUCTION.

After the plans had been drawn, the directors of the Home very carefully considered the matter of constructing the plant. The Engineer's estimate, which was very carefully made, and which took into consideration the isolated part of the country in which the work was to be done, showed that the construction would cost about $7,000. After due deliberation, the directors came to the conclusion that, by using institutional labor and stone from their own quarry, a reduction in this estimate could be effected. Nearly all of the work at the institution is done with its own labor, and it was decided to carry out this work in the same manner, and in fact all of the work, with the exception of the stone masonry, was done with institutional labor. The work was begun in the late summer of 1912, and completed in October of this year. The time of building was somewhat lengthy, but on considering that for three months during the winter of 1912-13, practically nothing was done and that during the spring and summer of 1913, all of the routine farm work was looked after, the record is highly commendable. No exceptional difficulties were experienced during the construction. The total cost of the plant complete has been somewheat less than $5,000, which means a saving to the Home of about $2,000.

The plant itself is located behind the brow of a small hill and is consequently invisible from the highway. It presents a very neat and unique appearance with its pointed stone masonry walls. Such construction with its incident beautification, would, if more generally carried out in this country, tend to overcome much of the prejudice existing against sewage treatment plants.

On the occasion of an inspection by the writers, in the late fall of 1913, the plant had been operating successfully for about two months. The effluent was clear, colorless, odorless and non-putrescible. No operating results were available at that time, although from unofficial reports, we believe that every requirement will be very fully met, and that operating costs will be very low.

Too great credit cannot be given to Mr. Davis Garrett, the superintendent of the Home, for his interest in this comparatively new and difficult work. His intelligent construction has made a plant whose beauty and stability furnish an admirable example of a modern up-to-date treatment plant.

The plant was designed by Mr. Chas. F. Mebus, M. A. S. C. E., of Albright & Mebus, Consulting Engineers of Philadelphia, Pa., and Mr. P. F. Mebus, C. E., Assistant Engineer. The construction was under the supervision of Mr. F. R. Berlin, C. E., Assistant Engineer.

$125 000 Dam at Cedar Rapids, Ia.—Plans and specifications are being prepared for the construction of a dam for the city of Cedar Rapids, Ia. It will be of the solid gravity type about 650 ft. long and will be provided with floodgates and with a small lock. The local railway and light company will pay a rental of $12,500 per year for the use of the dam, and will construct a power plant. At an election on Dec. 16 the city officials were authorized to issue $125,000 of bonds for the construction of the dam. Core borings have been made at the site, but it will be some time before the plans are ready for bids. Hedrick & Cochrane, Kansas City, Mo., are the engineers.

Some Important Features in the Design and Operation of Imhoff Sedimentation and Sludge Digestion Sewage Tanks.

An article entitled, "Some Observations upon the Preparatory Treatment of Sewage with Special Reference to California Conditions" was published in Engineering and Contracting of Dec. 10, 1913. That article, which discussed sedimentation and sludge digestion in single and two-story sedimentation tanks, independent sedimentation and sludge digestion, the odor problem in connection with Cameron and Imhoff septic tanks, and briefly reviewed California experiences in the preparatory treatment of sewage, was from a paper before the California League of Municipalities by Charles Gilman Hyde, Professor of Sanitary Engineering at the University of California and consulting engineer of the California State Board of Health. The present article is taken from a paper prepared by Prof. Hyde for "Pacific Municipalities," the official organ of the League of California Municipalities. We are indebted to Professor Hyde for a copy of the paper here given.

While the present article is written for conditions as they exist in California, practically all that is given is of general application. The climatic conditions in California have apparently developed certain special problems in the preparatory treatment of sewage in Imhoff tanks, as well as in septic tanks of the older Cameron type. Recent experience has demonstrated the need of more careful design, as well as painstaking operation. Unfortunately, careful attendance and wise operation have not as yet obtained in connection with any Imhoff tank installation in California of which the writer is aware. For this reason, it has been deemed important to summarize at this time the main conclusions with respect to certain features of design and operation which appear to be warranted by experience, both in general and in particular, under California conditions.

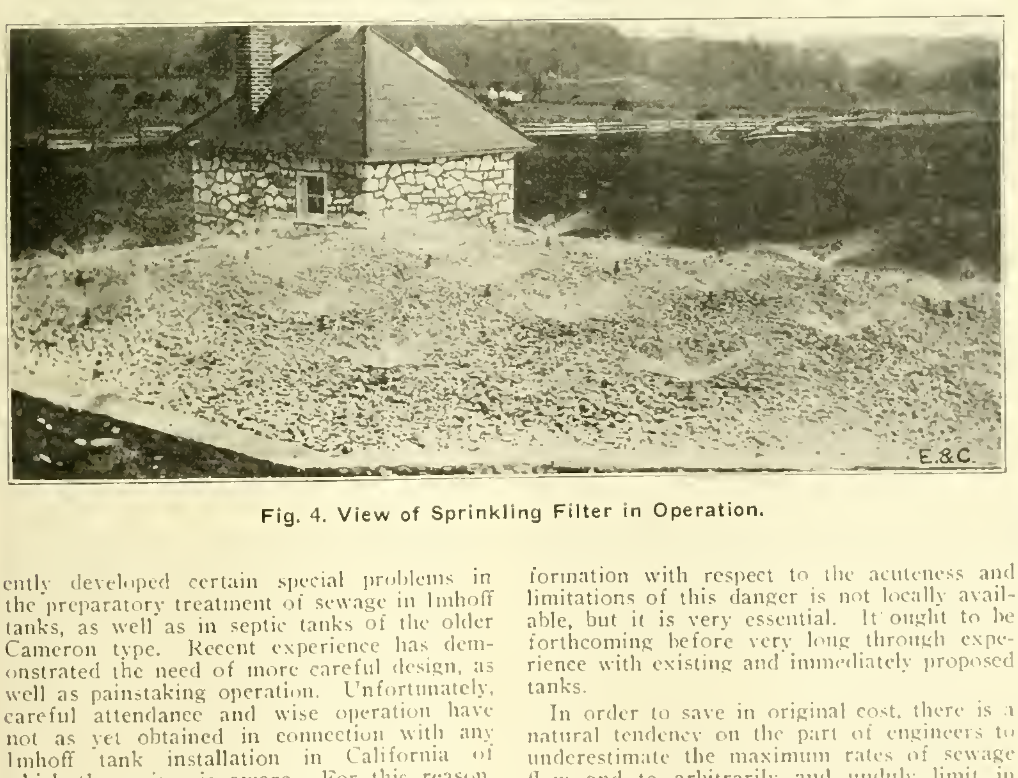


Fig. 4. View of Sprinkling Filter in Operation.

CALIFORNIA CONDITIONS AS AFFECTING SEWAGE TREATMENT.

(The following paragraph from the article published in Engineering and Contracting of Dec. 10, 1913, is quoted to co-ordinate the reader with California conditions as affecting sewage disposal.—Editors.):

Speaking generally, the climatic conditions in California are strikingly favorable to all processes of sewage treatment which involve biological activities, that is, the activities of bacteria, worms and other organisms which operate to decompose, digest, hydrolyze, gasify or oxidize the organic and mineral matters in sewage. The activities of the organisms in question are greatly affected by temperature, high temperatures tending to induce rapid development and great activity and low temperatures tending to inhibit development and to lower the vitality and the activity of the organisms. Except at the higher altitudes in California, freezing weather occurs but rarely and for short intervals, while over large areas of the state heavy frosts are only occasionally known. In fact, the conditions are such that sewage treatment works must be designed and operated with the expectation of extraordinary biological activities. In common septic and Imhoff tanks it is necessary to prevent excessive scum formation, and in the latter tanks, especially, large scum chambers must be provided readily accessible for frequent scum agitation, and, if necessary, scum removal. It is to be noted in this connection that scum formation is more or less directly related to biological activity and the production of gas through the decomposition of organic matter.

FEATURES OF DESIGN OF IMHOFF TANKS.

The Subsidence Period.—Fundamentally, the subsidence period provided in Imhoff tank installations must be sufficiently prolonged at most critical times (i. e., times of maximum flow or of the discharge of wastes which are most difficult to treat), to compel as effective subsidence as is reasonably possible. Strangely enough, the available evidence goes to show that the weaker sewages require as long, if not longer, periods of subsidence than do the stronger. At any rate, the "flowing-through" period for domestic sewages should never be less than, say, from 1 to 1½ hours. This means that the average period of subsidence should be considerably greater than this, and the maximum period, represented by minimum night flow, would be still further prolonged. With unclean sedimentation chambers and with too prolonged storage, there is evidence that some septic action may take place. Information with respect to the acuteness and limitations of this danger is not locally available, but it is very essential. It ought to be forthcoming before very long through experience with existing and immediately proposed tanks.

In order to save in original cost, there is a natural tendency on the part of engineers to underestimate the maximum rates of sewage flow and to arbitrarily and unduly limit in design the "flowing-through" period.

As a matter of fact, the subsidence or retention period, based on the total displacement volume of the "flowing-through" chambers in any installation, should not be less than, say, from 2 to 4 hours, varying with conditions, for the average rate of flow of sewage from a population estimated for a reasonable period in advance of the time of construction. These figures are larger than those which have been given for German conditions, but it is believed that local circumstances demand longer intervals of subsidence.

Special consideration must be given to the effect of pumping in those cases where pumps must be installed, for reasons of economy of construction, on the inlet ends of Imhoff or other subsidence tanks. With unduly large pumps operating intermittently at comparatively rapid rates, there is a tendency to force the sewage through the subsidence chambers without giving proper opportunities for the deposition of sludge.

If such pumps are employed, the volume discharged at any one operation would be only a fraction of the total volume or displacement capacity of the "flowing-through" chambers, and such chambers should be effectively baffled to break up the adverse currents which would otherwise inevitably be formed.

The Sludge Digesting Period.—The period provided for the digestion of sludge should be determined upon the basis of a number of considerations, among which are primarily the character of the sludge and the probable intensity of biological activity in the sludge digestion chambers. This latter factor is probably the more important one with domestic sewages. It is probable that a minimum period of two months is required for the complete digestion of sludge under optimum conditions, and that the total effective storage capacity of sludge digestion chambers should never be less than four months under California conditions in large plants. The smaller the plant the longer the period of sludge storage which may well be provided, because of the greater variability which is likely to obtain with respect to the character and condition of the sludge and because of the desirability of reducing to a minimum the attendance required to be given to the withdrawal and drying of sludge, etc. For the ordinary case in California, giving due consideration to climatic conditions, including the duration of the rainy season, it would appear that an effective sludge capacity representing six months of storage of the sludge received from the population upon the basis of which the tanks are proportioned would be reasonable.

The volume of sludge found at any time in a given tank depends, of course, primarily upon the period represented by the deposits. It also depends upon the character of the sewage, the period of subsidence, the activity of the organisms in the sludge, the volume of gas entrained therein and the water content. The last named factor is an extremely important one. Freshly settled sludge may contain from 90 to 95 per cent of water; a thoroughly digested sludge may contain from 75 to 80 per cent of water. With a given dry weight of solids, the ratio of the volume of a sludge containing 75 per cent moisture to that of one containing 95 per cent of moisture is as 1 to about 6.2. In other words, the volume of 75 per cent moisture sludge is only 16 per cent of 95 per cent moisture sludge.

The volume of thoroughly digested sludge (75 per cent moisture) from a system of sanitary sewers (street wastes and storm waters being, of course, excluded therefrom) is given by Dr. Imhoff as 0.1 litre per day per person connected with the system. This is equivalent to 0.0035 cu. ft. per person, or 3.5 cu. ft. per 1,000 persons per day. With shallow tanks and the less complete dewatering of the sludge which would ordinarily take place therein, the volume of sludge to be withdrawn would be greater. With combined sewers receiving storm wastes from which the heavier mineral matters have been removed in silt chambers on the outfall sewer lines, German experience indicates a volume of 0.2 litres of digested sludge per capita per day.

It is to be noted that the sludge present at any stated time in the lower chamber of an Imhoff tank is by no means uniform in composition. The uppermost portion of the deposit would, of course, be comparatively fresh, and therefore would have a very high water content. The lowest layers, if deposited several months previously, should be well digested and greatly reduced in volume, due to a very considerable dewatering. Between the top and bottom there is probably a gradual change in the composition and character of the sludge. To calculate the capacity of a given sludge digestion chamber, it is necessary to take into account at least these three factors: the amount of sludge to be deposited per day (perhaps best expressed in terms of thoroughly digested sludge), the average volume of this sludge as determined by the changes taking place during the storage or retention period, and the period of storage. Information is needed as to the water content of the sludge which would be found at stated times and under stated conditions at different depths in the sludge digesting chambers of Imhoff tanks of varying total depth.

Based upon the above considerations, Mr. Kenneth Allen has derived the following rough general formulas for the determination of the effective capacity of the sludge chambers of an Imhoff tank installation:

C (sanitary sewers alone) = 5.25 P. D.
C (combined sewers) = 10.50 P. D.

where

C = effective capacity of chambers in cubic feet;
P = population in thousands provided for; and
D = duration of storage of sludge or the retention period in days.

By the term "effective capacity" of sludge digesting chambers should be meant the volume measured from the bottom up to a point such a distance below the bottoms of the slots of the "flowing-through" chambers that no gas-laden sludge may be carried backward through the slots. This level is ordinarily from 18 to 24 ins. below the slots.

General Shape and Arrangements of Tanks.—Due to their very considerable depth (25 to 30 ft.) and the consequent heavy earth pressure involved, German tanks have consistently been made circular in form with conical bottoms, the "flowing-through" chambers being of both longitudinal and radial type. American engineers, except for the deepest tanks or for very unfavorable soil conditions, seem to prefer the rectangular type, although circular tanks have been installed in a number of cases; in fact, several of this type have already been built and others are projected in California. Circular tanks are favorable from the standpoint of their capacity for resisting earth pressure, but there can be no question but that they involve heavy additional expense for form work as compared with rectangular tanks. In some cases the expense so represented is greater than that which would be represented by the additional concrete and reinforcement possibly required for the rectangular tanks.

Questions of depth are important. The greater the depth the smaller the volume of digested sludge will be, due to the greater compression of the entrained gases (resulting from the decomposition of sludge), and due to the smaller water content or percentage of moisture. In populous communities, where land is very valuable and sewage treatment sites are restricted, as, for instance, in portions of the Emscher Valley in Germany, where the Imhoff tank was first developed, any saving in the size of sludge drying beds which may be effected, without excessive cost, would be welcome. But where land is comparatively cheap and where climatic conditions as related to the drying of sludge are very favorable, as is notably the case in California, the writer is inclined to believe that shallower depths than have been employed in Germany may be used with satisfaction and may represent a real total economy. It would appear that tanks from 16 to 20 ft. in total depth can be made to be satisfactory, provided that, in their design, proper account is taken of the greater average volume of sludge which will obtain with a given weight of dry solids in the shallower tanks, as compared with the deeper tanks to which most of the data now available relate.

In Germany, in installations involving several horizontal-flow units, the tanks are frequently arranged in pairs or groups, through which the sewage flow is reversed in direction from time to time, so that the character and amount of sludge deposited in the several tanks shall be uniform. This practice has been followed in a number of instances in America. American engineers, however, when employing horizontal flow rectangular tanks in installations involving several units, frequently design them independently of each other, the flow of sewage through each being determined by its size or capacity. In this way, since each tank receives its due proportion of the flow of sewage and hence each receives the same character of sewage and operates presumably under the same conditions, the sludge deposits in each should be similar in character and relative volume.

There seems to be no valid reason why multiple "flowing-through" or sedimentation chambers cannot be employed in a single Imhoff unit. Such an arrangement is, of course, primarily suited to rectangular tanks. There would apparently be no greater tendency for sewage to pass from sedimentation chamber to sedimentation chamber through the common sludge digestion chamber than there would be for an interchange of contents to take place between a single "flowing-through" chamber and the sludge digesting chamber beneath it. Experience has demonstrated that such a tendency, if it exists at all, is very slight and does not require special consideration. Theoretically, of course, due to relative temperature changes, such an interchange of liquids might take place. Where several sedimentation chambers in a single unit are proposed, great care must be taken accurately to proportion the flow of sewage between them by the use of suitable and sufficient inlet and outlet devices. Such devices, it is believed, can be readily arranged. In a recently proposed installation the writer has suggested as many as three sedimentation or "flowing through" chambers or channels above a common sludge digestion chamber. One tank has already been constructed in California with two "flowing-through" chambers of the type in question.

It should be remembered that the greater the width of such sedimentation chambers the greater their depth must necessarily be, and the greater will be the volume of ineffectual space in the scum chambers above the level of the slots. The use of multiple sedimentation chambers may often permit of their better proportioning and greater effectiveness.

It will be observed that there is an opportunity for the display of considerable originality in the design and arrangement of these structures, and it is to be hoped that effective means may be taken to simplify their construction as much as possible, at the same time maintaining their efficiency.

The "Flowing-Through" or Sedimentation Chamber.—Thorough diffusion of the sewage is essential to the highest efficiency and to the utilization of the nominal storage period as completely as possible. This means that for any given flow of sewage the actual retention period must approximately equal the nominal period as determined by the volume of the sedimentation chamber and the rate of flow. It is impossible in practice to secure absolutely complete diffusion, but it may be materially assisted in several ways. The sedimentation chamber may be made very long as compared with its width and depth; effective means may be devised for bringing the sewage into the chamber and taking the sewage from it through a number of ports or openings; and specially designed diffusion baffles may be used at the ends of the chamber, and, possibly, under certain conditions, distributed throughout its length. Preferably all of these features should be employed to secure effective displacement or diffusion of the sewage in passing through any sedimentation chamber.

Aside from the matter of the storage period above discussed, the rate of flow or velocity of forward motion through such chambers is very important. General experience derived in the subsidence of sewage and of water indicates that the average rate of forward motion should not, as a rule, be greater than from 50 to 75 ft. per hour. In Germany, in the sedimentation chambers of Imhoff tanks, the average nominal velocity is stated to be not usually less than 60 ft. per hour. It is wise to adopt a low average rate of forward travel in the design of chambers of this sort, because of the frequently unexpected rapid growth of our communities and the general tendency to underestimate the volume of sewage flow.

In order that the sludge settling from the sewage passing through these tanks may, to the greatest possible extent, automatically and without assistance pass through the slots into the digesting chamber below, it is important that the bottom slopes be not less than, say, 1.25 on 1.0. It is probably seldom desirable to increase these slopes beyond 1.5 on 1.0. The angles represented by these slopes are, respectively, 51° and 56°, measured from the horizontal. The interior surfaces of these chambers should be very smooth. Slots in baffles or other obstructions in sedimentation chambers should have their upper surfaces steeply sloped, so that deposits will not accumulate upon them.

The proper width of slots in the bottoms of sedimentation chambers through which the sludge may pass to the digestion chambers is determined more by experience and by certain practical considerations than by theory. From 6 to 8 ins. would represent good design for large chambers, while even 4 ins. may be found satisfactory for small works, where the sewage is screened or otherwise free from the danger of conveying extremely coarse matters. A most important detail is the overlapping of the edges of the bottom slopes of the sedimentation chamber so that no gases or gas-laden particles from the lower sludge digestion chamber may pass upward through the slots. The overlap, measured horizontally, should never be less than 3 or 4 ins. A safer figure would be 6 ins.

The Sludge Digestion Chamber.—Comments have been made above with respect to the capacity, depth and general shape of sludge digesting chambers. It remains merely to discuss the features of bottom slopes, sludge removal pipes and sludge stirring and diluting devices.

The bottom slopes of sludge digesting chambers may be comparatively slight. A slope of 1 on 2 would appear to be ample, and under certain conditions an even smaller slope may be desirable. It should be noted, however, in this connection, that earth and water thrusts are the better resisted by inverted cones or pyramids which are not unduly flat.

Sludge removal pipes are seldom less than 8 ins. in diameter nor more than 10 ins. In very small and shallow installations 6-in. or even 4-in. pipes may be employed with satisfaction. Sludge removal pipes should be enlarged, if possible, by some sort of bell end at their lower extremities. They should be set with their lower ends from 8 ins. to 1 ft. above the bottom, and they should be rigidly supported. The outlet ends of these pipes should be placed with reference to the normal water level in tanks so that the static head will be sufficient to force the digested sludge up through the pipes and out into the channels delivering upon the sludge drying beds. Experience has shown this necessary head to be from 3 to 6 ft., depending on conditions. On general principles, the head allowed should be as great as possible. Arrangements for backfilling these pipes with water or settled sewage after use in the removal of sludge should be made. The pipes should, in all cases, be heavily coated with materials which will resist the action of the gases and decomposing products in the sludge digesting chambers.

A perforated pressure pipe, arranged in the form of a ring, should be installed near the bottom of each sludge digestion chamber. It will serve a double purpose in that it may be employed to loosen the sludge should it become too dense and fail to flow readily through the removal pipe, and it may be used, if necessary, to dilute the sludge and carry away any by-products of the growth of bacteria and other organisms which may have accumulated to such an extent as to seriously lower the vitality and reduce the activity of these very necessary organisms. Such an unfavorable condition in the sludge would appear upon its withdrawal or from an examination of samples properly collected. This pressure pipe may be supplied with water or with settled sewage. If the latter is employed, it will ordinarily require to be pumped, and if such is the case, the pump suction should be provided with a fine screen. The holes in this perforated pipe should be suitably spaced and directed, and should be of proper size to secure the desirable results above outlined.

Scum Chambers.—The scum chambers, so called, in Imhoff tanks, really constitute a part of the sludge digesting chambers; in other words, their uppermost portion. In California, due to extreme biological activity, unscreened sewage ordinarily decomposes very rapidly, forming large volumes of gases, which lift the particles and produce scum instead of sludge. In some cases the amount and nature of the scum formation has represented a most troublesome feature of operation of septic tanks of the Cameron type and of Imhoff tanks. It is important that scum chambers be of generous horizontal area, perhaps at least 15 per cent of the total superficial area of the sedimentation chambers. They should be perfectly accessible, so that the scum may be frequently broken up with a view to liberate the entrained gases and permit the solids to settle to the bottom, where proper digestion may take place. If this is not done, very troublesome amounts of scum may be formed, and, under extreme conditions, practically the entire weight of solids deposited in a tank will be lifted into the scum chambers and will collect to a depth which will be limited only by the dimensions of the tank.

This general matter has not received hitherto the attention which it deserves. It should be realized that not only do the scum chambers represent an important feature of design of Imhoff tanks, but they are essential to the operation of such tanks, both as respects their mode of action and their manipulation.

The Sludge Drying Bed.—The sludge drying bed is a very necessary adjunct to the Imhoff tank. No installation of Imhoff tanks should be attempted without suitable provisions of this sort.

The necessary area of the sludge drying bed or beds in any installation is dependent upon the frequency of removal and the consequent amount of sludge withdrawn in any given case. Conditions of atmospheric dryness and of temperature are important factors governing the size of sludge drying beds, because they determine to a considerable degree the rapidity of the drying out of the sludge. Heavy rains not only moisten the sludge but tend to otherwise interfere with its drying out on account of the unduly rapid liberation of the gas bubbles which they bring about. Sludge from deep tanks, because of the usually smaller water content and the greater compression, and hence the greater volume of entrained gas, will require less area in drying beds than will the sludge from shallow tanks of similar capacity. In dry weather well-digested Imhoff tank sludge of the usual consistency, containing suitable quantities of gas, will dry out and become spadable in from three to six days upon beds of proper construction. Less well-digested sludge will require a longer period for drying.

The usual rule for determining the size of sludge drying beds in Germany in connection with Imhoff tank installations serving sanitary sewers only is to provide about 330 sq. ft. per 1,000 persons represented by the tank capacity. In America this figure may be accepted provisionally for tanks of the usual depth, and under average climatic conditions. In California, where the dry periods are more prolonged, it is possible that, in some cases, especially where the sludge digestion chambers represent a considerable period of storage, the sludge drying beds may be smaller. It would not appear safe, however, to construct such beds with areas less than 250 sq. ft. per 1,000 persons, represented by the nominal capacity of any installation.

Upon the sludge drying bed the gas-laden sludge quickly loses the major portion of its water content, partly by evaporation, but largely by drainage. A 75 to 80 per cent moisture sludge, as removed from the tanks, should thus be reduced in volume from 40 to 50 per cent and in water content from 25 to 35 per cent. It is evident, therefore, that sludge drying beds must be well underdrained. If the soils in which they are located are not naturally very porous and sandy, with a water table several feet below the surface, sub-drainage must be artificially provided. Occasionally beds may be made sufficiently porous by simple underdrainage with agricultural tiles. But where tight soils exist it will be necessary to construct tile underdrains superposed by 8 to 12 ins. of porous material graded from bottom up from coarse to fine. Construction details of such beds are available in engineering literature.

Covers Versus Screens.—From the operating program outlined herein it should appear that constant inspection of and frequent access to the sedimentation, scum and sludge digestion chambers of Imhoff tanks are oftentimes absolutely essential to the securing of satisfactory and effective results. Covers over these chambers, therefore, are to be avoided inasmuch as they are entirely unnecessary to the proper physical and biological action of the tanks and are a decided obstruction and hindrance to their proper manipulation. If covers must be provided they should be sufficiently elevated, with windows and doors in the side wall, so that all parts of tanks may be well lighted and readily accessible to attendants.

The experience available to date indicates that some sort of a screen of trees or a hedge around the site of an Imhoff tank installation would be much better than covers from several points of view. In the first place a hedge of California privet, eugenia, pittosporum, laurustinus or any one of a dozen other available plants would be infinitely more artistic than any cover could possibly be. If such hedges were allowed to grow up quite high they would aid in diffusing any odors generated by the plant (but such odors should not ordinarily obtain with properly operated Imhoff tank installations). Well-ordered hedges would do much to allay the suspicion of neighbors and passers-by, while a board fence or a cover would tend to arouse their suspicion and offend their aesthetic senses. The psychological side of the problem of nuisances and offense to the public is extremely important and should receive particular attention in certain cases. There is no excuse for unsightliness even in connection with such commonplace things as sewage treatment plants.

Royalties.—The Imhoff tank has been patented in Germany, in the United States, and elsewhere and installations are subject to royalty fees. These fees are comparatively small and are payable to the Pacific Flush Tank Co. of New York and Chicago, acting as American agents for Dr. Karl Imhoff.

Dr. Imhoff and his representatives do not attempt to design installations for communities, but require that local engineers shall design and construct such works. In this respect their program is quite different from that of the Cameron Septic Tank Co., which originally attempted to usurp the place of the local engineer and to design and install plant itself. Their attempted royalty charges have also been relatively excessive as compared with the charges which are being made in connection with Imhoff tanks. Dr. Imhoff is glad to advise engineers respecting special features of these tanks.

The schedule of charges, represented by a single payment when the construction of work is begun, varies with the population which the installation is proportioned to serve and is believed to be approximately as follows:

250 people	$ 25 (smallest fee)
500 people	50
1,000 people	80
5,000 people	255
10,000 people	400
20,000 people	740
30,000 people	960
50,000 people	1,500
100,000 people	2,500
200,000 people	4,200
500,000 people	9,000

OPERATING FEATURES OF IMHOFF TANKS

Suggested Schedule of Operations.—The following schedule of operations is tentatively suggested as essential to the satisfactory and effective working of Imhoff tank installations. This program is based upon general as well as local experience; but it is to be realized that experience may show that other features of manipulation may require to be taken into account. However, it is believed that, if the following procedure is consistently carried out, the results will be generally satisfactory:

(1) *Cleaning Sides and Slots of Sedimentation Chambers.*—Every few days the walls and sloping bottoms of sedimentation chambers should be gone over with a rubber squeegee (a flat hard-wood board from 12 to 24 ins. long and from 3 to 4 ins. wide with rubber edges and a light wooden handle of length determined by the dimensions of the chambers) and all adhering solid matters pushed down through the slots. Floating materials, such as matches, corks, etc., behind baffles and all scum collecting upon the surface of the sewage passing through the chambers, should be removed. This may be done by means of a dish-shaped perforated skimmer, perhaps 18 to 20 ins. in diameter, attached to a long wooden handle. The scum and floating material may be thrown into the scum chambers connecting with the digestion chambers. No decomposing material should be allowed to accumulate in these chambers, otherwise septic action may take place and disagreeable odors produced.

(2) *Reversal of Flow.*—In installations where two or more tanks are operated in tandem the flow should be reversed in direction through the tanks at least once a month. This is necessary or at any rate very desirable in order that the sludge collecting in the several tanks may be maintained as nearly as possible uniform in composition and volume.

(3) *Breaking Up and Removal of Scum in Scum Chambers.*—Every few days, depending upon the character and composition of the scum, the rapidity with which it is formed, etc., the scum collecting in the scum chambers should be thoroughly broken up with a rake or other suitable device and pushed down into the digestion chambers. If this is not faithfully and consistently done at sufficiently frequent intervals scum may collect to altogether too excessive depths in the scum chambers. Great depths of scum will not digest properly and may give off more or less offensive odors of decomposition. If, in spite of frequent and thorough agitation the scum layers continue to deepen, relief may be had by spraying them with a hose under pressure, thus breaking up the particles somewhat and liberating the entrained gas. If this is ineffectual and the scum layers become too deep, dense and hard they should be removed by means of shovels. The scum so removed during the ripening period should preferably be placed in a pit and covered with earth. After the process of digestion has become effectively established in the lower chambers, any scum removed from the scum chambers may, in some cases, be satisfactorily treated by placing upon the sludge drying bed and mixing with the digested sludge.

(4) *Observation of Rate of Deposition of Sludge.*—The rate of deposition of the sludge in the sludge digestion chambers should be carefully watched and the elevation of its surface should be determined as frequently as once in each week or ten days. This may be done by means of a weighted board or sheet iron plate from 12 to 18 ins. square attached to a wire or cord or to a long light rod of wood or gas pipe.

(5) *Removal of Sludge.*—When the surface of the sludge has risen to a point 18 or 20 ins. below the slots some of the thoroughly digested sludge at the very bottom of the tank should be drawn off. The amount withdrawn at any one time should not be too great. In the first place only well digested sludge should be removed. Again, it is better, at least in large plants, to remove relatively small amounts of sludge at frequent intervals than to withdraw very large quantities infrequently. If the entire volume of sludge were removed at any one time, the entire process of ripening or establishing effective biological action in the lower chamber would require repetition. This ripening period is sometimes marked by very excessive and troublesome scum formation. Great care must be taken not to allow the sludge to pass too rapidly out through the sludge withdrawal pipes because the less-well digested and softer sludge nearer the surface

might be forced by the water pressure out of the tank rather than the more compact, thoroughly digested and less-readily flowing sludge at the bottom. In other words, an inverted cone of least resistance might be formed and eventually only liquid, rather than solids, would pass from the tank.

(6) *Use of Perforated Pressure Pipes.*—If the sludge should not flow readily through any sludge removal pipe it may be loosened by forcing clarified sewage or water through the perforated pressure pipe in the bottom of the digestion chamber. If clarified sewage is employed it will require to be pumped by a hand or motor-driven pump whose suction should be guarded by a brass screen having at least ten meshes per linear inch, so that the perforations in the pipe may not become clogged. According to Saville, excessive scum formation may sometimes be checked by the occasional use of this perforated pipe. When this is done some of the gas in the sludge is driven off and there is consequently less tendency for the sludge to rise into the scum chambers. Furthermore, if the activity of the organisms in the sludge seems to be retarded, due to the excessive formation of so-called enzymes or other by-products of their existence, these may be disseminated and diluted by spraying fresh clarified sewage or water through the perforated pipes in question.

(7) *Refilling of Sludge Pipes.*—After each withdrawal of sludge the sludge pipe should be backfilled with water or clarified sewage. If the latter is employed it may be pumped into the pipe by means of the force pump mentioned in (6). If the pipe were not thus backfilled it would almost certainly become clogged with dried-out sludge. If, from any cause, the sludge pipe should become clogged, it may be cleaned by pushing a rod through it or by forcing water or clarified sewage through it under sufficient pressure. In order that these methods of cleaning may be utilized the straight run of the pipe should be continued to a point slightly above the high water line in the tank. This portion of the pipe may be capped if desired, the cap being fitted with a pump connection.

(8) *Drying of Sludge.*—The depth of sludge deposited upon the drying bed at any one time should not be over 12 or 15 ins. In dry weather the sludge should become spadeable in a few days and should be removed from the bed. It may be used as a fertilizer upon lands or it may be used for filling in low places upon which it is not anticipated that houses will ever be built. German experience indicates that such material is a fairly satisfactory fertilizer and is well worth hauling away and utilizing upon cultivated lands. It has the advantage of being able to lighten heavy soils, and it would doubtless furnish humus to soils which are lacking in this feature. When deposited in fills the thoroughly digested sludge does not undergo putrefaction and will not cause any nuisance due to odors.

(9) *Resurfacing of Sludge Drying Bed.*—Whenever the surface layer of the sludge drying bed becomes clogged or is taken away in the removal of the sludge a new layer should be applied. This feature is important in order that water draining from the sludge may be removed quickly either by seepage into the underdrainage system or into the naturally porous sub-surface materials, as the case may be.

(10) *Sampling and Testing of Sewage.*—The superintendent of sewers or other official in charge of the sewage treatment and disposal works should be provided with Imhoff conical glass graduates for testing the amount of suspended matter in the raw and settled sewage. From these observations the efficiency of any installation may be roughly determined. The results should be recorded on proper blanks prepared for the purpose. Ordinarily this testing should be done once a day, but occasionally it should be done throughout the day in order to determine the variations in the composition of the sewage.

(11) *Measurement of Sewage.* Every installation should be provided with a measuring weir, suitable orifices or other devices for determining with reasonable accuracy the rate of flow of sewage. Whenever possible a recording, self-integrating gage should be installed to exhibit the flow of sewage at all times. This gage should be systematically checked against a suitable glass gage and scale or, better, against a hook gage.

Investigations Preparatory to the Design of a Comprehensive Sewerage System for Cincinnati, O.

Within the next few years a comprehensive sewerage system, costing several million dollars of which $3,000,000 already has been voted in bonds, will be installed in Cincinnati, Ohio. The various major features of the proposed system were discussed and the appropriate quantitative data were given in an article published in the Current News section of Engineering and Contracting for Jan. 14, 1914. The present article describes the preparatory investigations requisite to a comprehensive improvement of the sewerage conditions of the city. The information given is taken from the latest annual report of Mr. H. S. Morse, engineer in charge of the division of sewerage. This sewerage investigation is now nearing completion and its details will be fully described in a published report of 700 pages now in the hands of the printer.

The work on the sewerage investigations was begun in April, 1912. After an inspection of existing data and a conference with the consulting engineer, Mr. Harrison P. Eddy, who is retained for this work, the preliminary investigations were divided into these sections: underground survey, topographic survey, new and relief sewerage, intercepting sewers and creek improvements and sewage treatment. As descriptive details of these various sections of the work are contained in the annual report for 1912, they will be taken up now only in a general way for the purpose of showing progress.

Underground Survey.—Upon examination of records of the city sewers, it was at once evident that they were incomplete and of little value in the careful planning of sewerage improvements. A survey of all existing sewers was immediately started and it was deemed advisable to locate not only the sewers, but surface investigations of all underground structures, as water and gas mains, telephone and telegraph conduits, etc. Field work was completed Nov. 21, 1913, 496 miles of sewers having been located. This total will be increased by the addition of several miles of sewers now under construction.

The data supplied by this survey are being plotted, and traced on sheets 24x34 in. on a scale of 50 ft. to the inch, excepting in the congested down town section of the city, where a scale of 40 ft. to the inch is used. These plats will be reproduced on white cloth prints for general use. Three hundred and ninety-seven miles of sewers have been plotted on 557 sheets, and if the present rate of progress is maintained, all plotting should be completed by June 1914.

Topographic Survey.—The purpose of the topographic survey is to provide an accurate map showing in detail the topographic and drainage features of the city. Although the area of the city is approximately 70 square miles, in order to include watersheds affecting the local problem, the area finally mapped was extended to a total of 102.16 square miles. All field work for the map, including a system of triangulations of extreme accuracy, 150 miles of precise level lines from which were established 217 permanent bench marks referred to mean sea level as a datum, 285 miles of transit traverse control and 102.16 square miles of plane table topography, was completed Nov. 3, 1913. A booklet has been published giving the descriptions and elevations of the 217 bench marks.

The details of this survey are shown on 48 sheets, 24x31 ins., the scale being 400 ft. to the inch and the contour interval 5 ft. These sheets have been inked in colors as permanent records, and in order to make these data available for use in various city departments, reproductions are being made by means of engraving, the work of engraving and printing of the reproductions being done by contract. Engraving is completed for approximately 11.5 square miles of area, and the entire work of reproduction should be completed in June 1914.

Care in the reading of proofs from the engraved plates is essential to the accuracy of the map, as the high standard maintained in the making of the map can easily be lost in the engraving. This work of proof reading, therefore, must be continued in a careful and painstaking manner to the completion of the work of reproduction.

New and Relief Sewerage.—Preliminary to the design of new or relief sewerage, studies were made of the rainfall records in Cincinnati from 1871 to 1912, furnished by the U. S. Weather Bureau, and a rainfall curve for Cincinnati was plotted from these records. This curve is now being used on all sewerage design being done by the city. Also it was thought advisable to start observations, which, if continued over a period of several years will furnish excellent data as to the coefficient or proportion of runoff of rainfall in Chicinnati. As this coefficient varies with the topography and the proportion of developed or impervious territory in a given area, four drainage districts of different characteristics were selected for study, namely, Walnut Street, Clark Street, Badger Street, and Bloody Run. These drainage districts have been carefully measured and analyzed, and the proportion of impervious area computed, and 13 maximum sewer gages have been placed in the sewers in these districts. In connection with the gages in the Bloody Run district, an automatic rain recording gage has been placed on the roof of a building. The Weather Bureau records of precipitation are available for comparison of rainfall and runoff in the Walnute Street district. A regular inspector has been assigned to make frequent visits to these various gages to obtain the records of gage heights and to keep the gages in order. The results from the automatic gages have been entirely satisfactory and as a rule the maximum gages have given good results where extreme velocities do not exist. This condition obtains in the lower end of the portion of the Bloody Run district now under observation and a variation in the type of maximum gage for this district must be devised.

The relief work has been so arranged as to attack first the problems of localities generally known to need relief and is being extended so as to include the entire city sewerage system. As rapidly as essential data are supplied by the topographic and underground surveys, computations of the existing and required capacities of the sewers are made and when found inadequate through calculations, house to house canvasses are made to verify the findings. Plans for relief of such districts are then worked out for use as the basis of future sewer construction. The character of this work is such that progress is necessarily slow, for the problems must be worked out by a few men particularly fitted for the work. As a rule, there are several methods of relief for any one locality and when sewers cost from $2 to $20 per foot, care and time spent in analysis and comparison of these methods result in the saving of thousands of dollars.

While, due to the lack of funds, the entire area of the city cannot be covered by these studies as a part of this investigation, this work should be continued without interruption as a part of the current sewer design work. By this means a plan will be provided which will show all trunk sewer lines needed to serve the entire area of the city whether developed or not. Then, as development calls for construction, such construction can be made a part of a comprehensive scheme for sewerage of the city sufficient for ultimate population and thereby prevent damages caused by inadequate sewerage and also the expense of frequent rebuilding of sewers.

Intercepting Sewers and Creek Improvements.—The work outlined for this section of the investigation included studies and reports leading to the relief of the unsanitary conditions existing in Duck Creek, Mill Creek and along the Ohio River front of the city, and reports and preliminary plans for each of these intercepters has been completed. As it was early decided that all sewerage planning should be based on the development of the city in 1950, studies of the probable area and population of Cincinnati in 1950 were made, with the conclusion that the area would be extended from 70 to 141.5 square miles and the population increased from the present figure of 384,000 to 720,000.

Based on these assumptions, a distribution of the population over the entire area was made, with further assumptions as to the particular sections to be devoted to residential, park, cemetery, industrial and commercial purposes. While the results of this work are largely conjectural and open to wide variations, they have sufficient grounding on past statistics of this and many other United States cities, as well as a reasonable allowance for the numerous agencies which may affect the growth of the city, to furnish assurance that intercepting sewers designed on these data will be sufficient for the requirements of the population of Cincinnati in 1950.

In addition to these data, preliminary to the design of intercepting sewers, information was needed as to the quantities of sewage for which provision should be made for the various classes of development. Consequently, gagings were made on eight sewers in mercantile districts, four sewers in industrial areas, and on two sewers in residential districts. In the majority of the sewers gaged, the readings were taken for a period of one week. With the completion of these preliminaries, the following units of maximum flow were adopted:

Water supply returned to sewers—135 gals. per capita per day.
Ground leakage into sewers—750 gals. per acre per day.
Industrial wastes—9,000 gals. per acre per day.
Commercial wastes—40,000 gals. per acre per day.

For use in the preparation of estimates, available information was collected and a report on the sub-surface materials of Cincinnati was made. (The various intercepting sewers were described in sufficient detail for the present in the article to which reference has been made.—Editors.)

SEWAGE TREATMENT.

Studies bearing on the question of the ultimate disposal of Cincinnati sewage are under the direct supervision of the consulting engineer, Mr. Harrison P. Eddy, and the report on this branch of the investigation has been completed.

As a basis for the report, a detailed sanitary survey including dissolved oxygen tests, was made of the Ohio River between the intake of the water works to the Fernbank dam, of the Little Miami River, Licking River, Mill Creek, Muddy Creek, and the Miami and Erie Canal from Hamilton to Cincinnati. Samples of water for chemical and bacterial analysis were collected from all of these streams. The sanitary survey of the Ohio River was extended either by correspondence, personal visits, or reports of the several state boards of health from Pittsburgh, Pa., to Warsaw, Ky.

In order to ascertain the character of sewage which may require treatment, a general investigation of the industrial wastes of 345 manufacturing plants in Cincinnati was made. These included tanneries, breweries, slaughter houses, packing houses, soap factories, galvanizing works, distilleries, glue, grease and oil works. Of this number, 132 plants were studied in detail. Twenty samples, representative of the wastes from these plants were collected and analyzed.

The report on Sewage Treatment in a similar manner to the closely related report on the Ohio River Intercepter does not indulge in untimely definite recommendations, but discusses thoroughly the local problems, presenting the findings for such future action as necessity may demand.

Object and General Features of the Sewage Treatment Experiment Station at the University of Michigan.

Last spring we learned that an experimental sewage treatment plant was to be installed at the University of Michigan and we expressed our enthusiasm over the possibilities of the undertaking in an editorial entitled: Experimental Work in Sewage Treatment at State Universities, which was published in our issue of April 23, 1913. At that time we spoke only in general terms of the contemplated Michigan installation, for the reason that direct publicity was not then desired by those who had the planning of the station in hand. The present article describes the object and general features of the experimental plant. Our information is taken from an article by William C. Hoad, Professor of Sanitary Engineering at Michigan, published in the bulletin of the Michigan State Board of Health for November, 1913.

Last year the regents of the university established a sanitary experiment station in the department of engineering, for the purpose of providing means whereby various problems related to public sanitation might be studied. The station building and the structures that comprise the physical equipment of the plant are now just completed, and the plant is being put into operation. It is located on the northerly slope of the campus, where the topography permits the compact and efficient arrangements of the various station structures.

The plans at present under way relate largely to the problems involved in city sewerage, especially the treatment and proper disposal of sewage, and of other city wastes; and to the protection of streams and lakes from pollution. It is the expectation, however, that opportunity may be found later for the investigation of certain problems of water supply and of water purification.

For the purpose just outlined, the station is equipped with fair sized working units of all the more reliable and better known sewage treatment devices, together with suitable apparatus for testing the results of the various processes. Some of the more important of these devices may be mentioned briefly.

THE SEPTIC TANK.

The septic tank is one of the best known and most valuable devises for the treatment of city sewage. When used alone it does not bring about a complete but only a partial purification of the crude sewage. Its action is to separate out the suspended solids of the sewage from the water in which they are contained. and afterwards to disintegrate and liquify these removed solids. The use of the septic tank in this manner will minimize the tendency of sewage discharged into lakes or sluggish streams to form objectionable deposits near the sewer outlet, and in general will permit the sewage to be discharged into smaller streams or bodies of water than would otherwise be possible without giving rise to local nuisances. The septic tank attains its greatest usefulness, however, when employed in combination with a filtration process, the septic tank removing and disposing of the sludge and the filter taking care of the impurities in solution in the sewage.

A good sized septic tank unit has been installed as a part of the station equipment, which will not only illustrate the septic tank process, but will in addition permit special studies to be made of such subjects as sludge disintegration, gas formation, efficient inlet and outlet arrangements, effective means of controlling the flow of sewage through the tank, most profitable detention period, and the volume and character of the residual sludge, and the best means of disposing of it.

THE IMHOFF TANK.

The Imhoff tank is a relatively new device for the separation of the sludge from sewage and the digestion or disintegration of its organic constituents The new device, although promising well for the future, needs to be rigorously tested out under American conditions before its advantages as well as its limitations can be fully known. The unit tank included in the experiment station equipment is carefully designed with this purpose in view. With it studies can be made of sludge digestion, the most efficient period of flow for the crude sewage, the most profitable detention period for the sludge, the nature and extent of gas formation, effective means of drying, and finally disposing of the residual sludge and other questions bearing on the economic design of large scale plants.

VARIOUS FILTERS.

Contact Filters.—One of the most adaptable and reliable engineering devices for the purification of sewage is the contact filter. In this device the sewage is very thoroughly clarified and is subjected to the action of certain scavenger forms of bacteria, which are colonized in immense numbers in filter beds of gravel or broken stone. Inasmuch as filters of this type are usually constructed from local materials, and since climatic conditions have an important bearing upon the efficiency of their operation, a total of ten filter units have been installed for the purpose of testing out local gravels of various sizes, broken stone, coke, screened cinders and any other promising material that may be found, and of experimenting with different methods and rates of operation.

Trickling Filter.—The trickling or sprinkling filter, now common in England and steadily winning its way in America, is peculiarly susceptible to climatic influences, being especially sensitive to extreme cold. A full-sized unit of this type of sewage filter is included in the station equipment. This filter is set out in the open, where it will be subjected to the rigors of the Michigan winters, and is so arranged that three kinds of filtering materials and two different depths of each material can be tried out at the same time under identical conditions. The connections are so arranged that the effluent from either the septic tank or the Imhoff tank can be applied to this filter.

Sand Filters.—Extensive areas in Michigan are covered by the drift formation, which furnish excellent opportunities for the purification of sewage by intermittent filtration through natural beds of sand. This is the method of sewage purification most commonly employed in Massachusetts, where the idea was originally developed; and the Massachusetts conditions with respect to topography and soil formation are in many ways similar to those existing in Michigan. The bacterial efficiency of the intermittent filtration process is so high and its general operating results so satisfactory that it should be given a thorough trial.

Two sand filters are included in the experiment station plant by which tests may be made of different qualities of sand or of sand and gravel; of most profitable rates of filtration, and of best methods of operation during the winter season.

OTHER PROCESSES AND DEVICES.

In addition to the foregoing, facilities are provided for studying methods and results of screening crude sewage through fine-mesh screens of various types; the sterilization of sewage and of the effluents from the various tanks and filters; the beneficial effects of aeration as an aid to other purification processes; the separation and concentration of sludge by plain sedimentation in specially designed tanks; and other special methods of treating the crude sewage, or of handling the sludge.

RELATION TO THE STATE.

The population of the cities of the state is increasing. With this growth of the cities and with the higher sanitary standards that have become prevalent in recent years, the difficulty of securing acceptable municipal water supplies has vastly increased, and this tendency may fairly be expected to continue. The cities must have adequate and suitable water supplies, however, at whatever cost. There is no handicap to a city's normal development more powerful, and no burden more heavy to bear, if the whole story could be told, than an inadequate and unsuitable public water supply.

Moreover, as the sewered population upon the drainage areas of streams and lakes increases and as industries grow, inevitably there comes about an increased pollution of the water, unless this is prevented by vigorous and well-directed remedial measures, such as the proper treatment of the sewage or industrial wastes before they are discharged into the stream. But sewage treatment costs money; both to build the disposal plants and to operate them after they are built, and both the cost and the efficiency of the treatment process depend to an important extent upon local conditions of climate, topography, geological formation, sewage characteristics and the like. One of the objects of the station will be to try out under the conditions existing in southern Michigan such sewage purification processes and devices as have demonstrated their usefulness elsewhere, and to exemplify these to city officials and to local health officers in a small plant operated under careful supervision.

The growing sensitiveness of the courts to questions of stream pollution, and the steady improvement of our everyday standards of common fairness and decency, no less than the slowly increasing actual pollution of our streams, are bringing many cities face to face with the problem of the proper disposition of their municipal wastes. It is the hope of the experiment station that it may be able to assist the cities of the state somewhat in meeting this problem.

Bridges

Concrete Tile Floor Construction for a Reinforced Concrete Highway Viaduct.

(Contributed.)

A new type of floor construction was adopted for the Trent Road Viaduct, which has recently been constructed in Spokane Co., Washington. This construction is somewhat similar to the combination terra cotta tile and reinforced concrete construction commonly used for the floors of buildings, except that a special hollow, rectangular concrete tile is used. This type of construction weighs about one-half as much as a solid concrete slab of equal strength, and it can be used on spans as great as 40 ft., for either buildings or bridges.

Figure 1 shows a cross section of the Trent Road Viaduct and indicates the type of construction used. This drawing also shows the heavy collision piers which were used to protect the columns in case of the derailment of trains passing under the bridge.

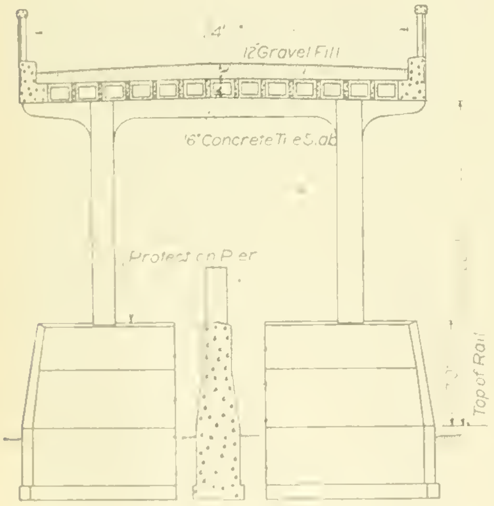


Fig. 1. Cross-Section of Trent Road Viaduct Showing Type of Construction.

Figure 2 shows a perspective view of a portion of the floor and illustrates its construction. It will be noted that the tiles are laid in rows, with spaces between the rows. Reinforcing bars are then placed near the bottom of these spaces, and the spaces are filled with concrete and leveled off flush with the top of the tiles. The forms required for this construction are known as "open centering," consisting of 8-in. plank laid under each joist, spaced 21 ins. on centers. Each tile has three beveled edges on one end, which form V-shaped joints between the ends of the tiles composing each row. The transverse temperature rods are placed in these joints which are then filled with the grout-like portion of the concrete, at the time that the joists are being poured.

The wedge shaped lugs, shown on the sides of the tiles, not only prevent their vertical displacement but when the spaces are filled with concrete transfer bending stresses from the concrete joists to the tiles; thereby causing the latter to act as the compression flange of the slab. The compressive stresses thus transmitted from the joists to the tiles are in turn transferred from tile to tile by the V-shaped grouted joints. From the above it will be seen that no poured concrete is required on top of the tiles, as is the case of terra cotta tile and concrete slab construction, in which the tiles act mainly as a filler.

The advantages of having no poured concrete above the top of the tiles are two-fold: In the first place, the top of the tile forms a strong and convenient working surface, making it possible to place all reinforcement, including the top bars, before any concrete is poured, and as all of the reinforcement is below this working surface there is no danger of it being displaced by the men who are wheeling concrete. The second advantage is that the concrete tiles insure a uniform and proper depth of slab.

The theory of combined action of joists and tiles was demonstrated by tests made in Spokane under the supervision of Mr. A. D. Butler, assistant city engineer, in which a concrete tile slab having the same cross-section and reinforcement as a hollow monolithic slab was shown to be 11 per cent stronger. Another proof of the strength and rigidity of this construction is to be found in the traffic test given to the Trent Road Viaduct before its acceptance by Mr. J. W. Strack, county engineer. The 16-in. concrete tile floor, on a span of 22 ft. 3 ins., sustained 150 lbs. per square foot of gravel fill, together with a concentrated load consisting of a 16-ton road roller, with a deflection at the center of the span of only $\frac{1}{40}$ in., which is less than $\frac{1}{10,000}$ of the span. The bridge was designed for 100 lbs. of fill and a 15-ton road roller.

The cross-section and reinforcement required for any given load and span is determined in the same manner as though the construction were poured monolithic, which is a conservative assumption. The lugs on the sides of the tiles are proportioned to take shear and bearing in the same manner as rivets in a plate girder, standard dimensions being adopted which place these stresses well within safe values. The lugs are made wider at the top than at the bottom to prevent any possibility of vertical displacement of the tiles.

The process of manufacture of the concrete tiles is like that used for ordinary wall blocks. The mixture used is composed of 1 part cement to 6 parts of well graded aggregate, the latter consisting of about equal parts of fine and coarse sand and screened gravel or crushed rock not exceeding ½ in. in size. The materials are mixed just damp enough to compact well under iron tampers. A metal-lined form and core are used for molding the tile. The tiles are formed on hollow rectangular pallets which fit over the core. The core is provided with a draw of ⅛ in. in the length of a tile so that after the materials are tamped in place it can be lowered through the pallet. The outer collapsible form, which is hinged to the working bench, is then folded outward and downward leaving the tile resting on the pallet. The tiles are then stored under cover for five or six days, during which time they are sprinkled twice a day. The most convenient length in which to manufacture and handle concrete tiles has been found to be 8 ins. Two men using the mold and core described above, with materials supplied to them, can manufacture about 150 concrete tiles per day. In manufacturing the concrete tiles for the Trent Road Viaduct a temporary storage shed was erected, the lumber of which was afterwards used for the centering of the bridge.

In the construction of concrete tile floors the end tiles in each row are closed by casting in them a 1½-in. layer of concrete, which prevents the poured concrete from flowing into the ends of the rows. As the open cell is each tile is more than four times as large as the corresponding cell in a terra cotta tile, this construction is well adapted for carrying pipes and conduits.

The use of concrete tiles for the Trent Road Viaduct reduced the weight of the structure 140 lbs. per square foot from that of the solid slab design originally contemplated. The floor construction described above was invented by White & Co., bridge and structural engineers, Spokane, Wash.

Results of Tests of a 40-Ft. Reinforced Concrete Highway Bridge.

There are few data on the actual strength of reinforced concrete structures, due mainly to the difficulty of making and analyzing tests. Strength and safety are of prime importance, but it is not good engineering practice to place too much excess material in our structures, as the cost will then greatly restrict their use. It is only by means of carefully conducted tests, followed by an intelligent analysis of the data obtained, that we can hope for a better knowledge of the actual strength of our reinforced concrete structures. The following article has been abstracted from a paper by Mr. D. A. Abrams, in Proc. American Society for Testing Materials, vol. xiii, 1913. The test reported is quite complete, and the data have been carefully analyzed. We have greatly condensed the material contained in the original paper, but have included practically all of the data of direct value to designers and builders.

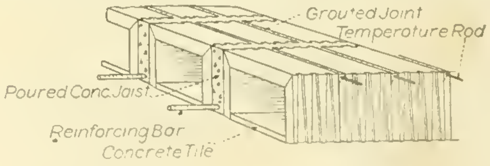
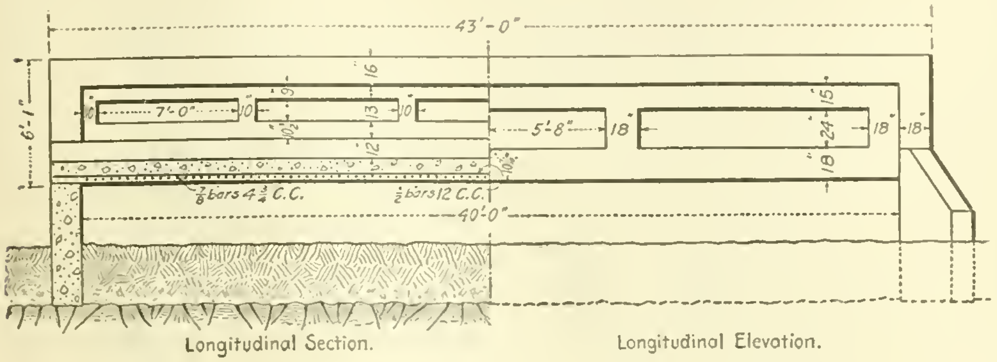


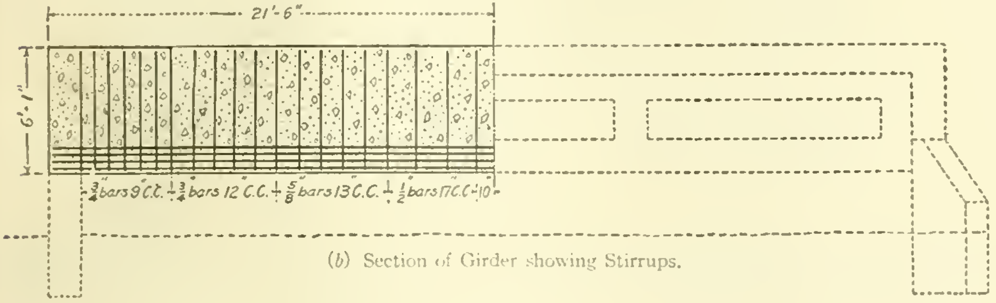
Fig. 2. Perspective View of a Section of the Floor Construction Used for the Trent Road Viaduct.

An exhaustive study of the different types of short-span highway bridges by Mr. A. N. Johnson, engineer, Illinois Highway Commission, led to the design of a through girder bridge with a suspended floor. A bridge of this type has obvious advantages for roadways of ordinary width, and is peculiarly adapted to the conditions in Illinois, since it gives a maximum water-way in locations where headroom is limited. Several hundred bridges of this form have been built through the co-operation of the city or township road commissioners and the Highway Commission, and all are now giving satisfactory service. The spans thus far constructed vary in length from 24 to 60 ft.

In order to furnish an object lesson for the benefit of skeptical advocates of older types of highway bridges, it was determined to make a test on a bridge of this kind by subjecting it to a load much more severe than it would ever be expected to carry in service. It was also felt that in this way valuable information could be obtained as to the action of the bridge under unusual loading, and thus secure a check on certain details of the design. Accordingly, in 1907, a bridge of 40-ft. span and 18-ft. roadway was built for testing purposes, from the design that was then being used. The test bridge was built inside the prison yard at the Southern Illinois Penitentiary, near Chester. All labor of constructing and testing the bridge was performed by convicts.



(a) Longitudinal Section and Elevation.


21'-6"
6'-1"
¾ bars 9 C.C. – ¾ bars 12 C.C. – ⅝ bars 13 C.C. – ½ bars 17 C.C. – 10

(b) Section of Girder showing Stirrups.

Fig. 1. Longitudinal Sections and Elevation of Reinforced Concrete Test Bridge.

Mr. A. N. Johnson is responsible for this design and for planning and carrying out the tests described below; and the work of building the test bridge and conducting the tests was done under his direction, the writer being indebted to him for information concerning the construction of the bridge. Mr. N. E. Ensign, instructor in the University of Illinois, gave valuable assistance in making the final load test and in compiling the data.

DESIGN AND CONSTRUCTION OF BRIDGE.

Design of Bridge.—The test bridge was designed for the usual loading of 125 lbs. per square foot, uniform load, or a 21-ton traction engine load distributed over an area 10 by 12 ft. The girders were 21 ins. wide on top and had an over-all depth of 6 ft. 1 in. Ornamental paneling, both inside and outside, served to relieve the severe lines of the design. The floor consisted of a 12½-in. reinforced concrete slab, suspended from the lower portion of the girders, and built monolithic with them. The abutments were of the low wing-wall type of plain concrete, 18 ins. thick. They rested directly on the rock floor of an abandoned quarry. Details of the design are shown in Figs. 1 and 2.

The longitudinal reinforcement of each girder consisted of ten 1¼-in. cold twisted square bars, placed horizontally throughout their length in four layers, as shown in Fig. 1. The depth of the girders from the top to the centroid of the longitudinal steel was 5 ft. 6 in. The webs of the girders were reinforced with vertical stirrups of ½ to ¾-in. cold twisted square bars, spaced as shown in Fig. 1 (*b*). These stirrups were bent in a U-shape and they engaged all the longitudinal bars and extended to the top of the girder. The webs of the girders were further reinforced by carrying each third bar from the cross reinforcement in the floor up into the girder, as shown in Fig. 2. In the design, the girders were assumed to carry the entire dead and live load as simple reinforced concrete beams under a uniformly distributed load.

The bridge floor was suspended from the lower portion of the girders as indicated above and rested directly on the abutments at the ends of the bridge. The principal floor reinforcement consisted of ⅞-in. cold twisted square bars, 4¾ ins. apart, placed across the roadway at right angles to the girders. The ends of these bars were disposed as shown in Fig. 2; ½-in. twisted square bars, 12 ins. apart, parallel to the girders, served to distribute the floor loads and to take temperature stresses.

The bridge, exclusive of the abutments, contained 97 cu. yds. of concrete and weighed about 206 tons.

Construction of Bridge.—The concrete was [illegible] Mississippi River [illegible] and a hard grade of crushed limestone from the prison crusher, in sizes varying from ¾ to 1¼ in., formed the aggregate. The concrete was mixed by hand. All reinforcement consisted of cold twisted, mild steel square bars. As the bars could not readily be obtained in lengths of 43 ft. for the longitudinal girder reinforcement it was necessary to weld all the 1¼-in. bars at about 10 ft. from one end. The welds were alternated when the bars were placed, the welding being done at the prison shop.

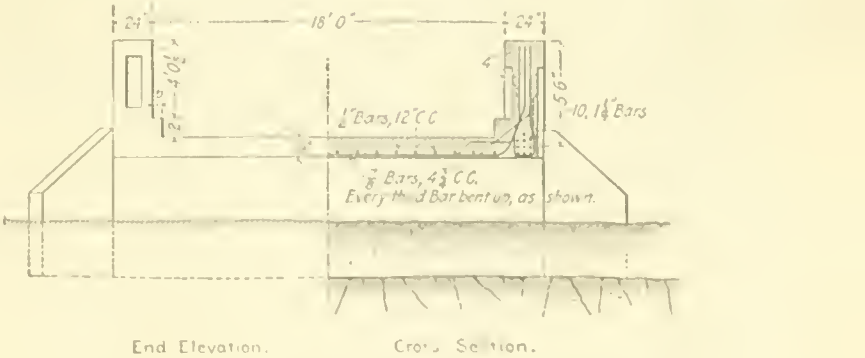


Fig. 2. End Elevation and Cross-Section of Test Bridge

The quality of the concrete in this bridge was apparently decidedly inferior to that obtained under ordinary field conditions, as the work was done by convicts without previous experience in this kind of work.

The concrete was placed Dec. 26 to 31, 1907, the temperature during construction varying from 54° to 28° F. On Dec. 31, the temperature was as low as 28° during the forenoon, and stood at about 35° during the afternoon. The girders were sufficiently protected to show no frost action, but scale was formed over certain areas of the floor, which later peeled off to a depth of about ¼ in.

PRELIMINARY LOAD TESTS.

Two preliminary load tests were made. From May 5 to 10, 1908, when the bridge was about four months old, a load amounting to 106 tons, equivalent to 296 lbs. per square foot, was placed on the bridge floor and was allowed to remain for three weeks. This load consisted of about 12 ins. of crushed stone, upon which blocks of cut stone weighing 100 to 200 lbs. each were piled. According to the constants used in the design, this load should produce a stress in the longitudinal steel of the girders of 16,000 lbs. per square inch. The deflection of the girders under this load was about 0.04 in. The total deflection of floor and girder was about 0.12 in.

In June, 1909, when the bridge was 1½ years old, a second preliminary load test was made. A track was built and provision made for pulling a loaded freight car on and off the bridge. The car was loaded with stone in such a way that a load of 44 tons came on one truck. The track was laid in 12 ins. of ballast, but the load of 44 tons was the only load that affected the measurements taken in this test. In order to concentrate the load, a separate piece of track 5 ft. long was placed in the middle of the span to receive the loaded truck. Deflections in the floor and girders and deformations in the girders were measured with the car on and off the bridge. The girder showed a center deflection of about 0.025 in. due to the load of 44 tons, but the temperature change with the load on or off the bridge was found to be about 0.02 in.

FINAL LOAD TEST.

Preparation.—Before beginning the final load test several brick piers were built under the girders and floor for use in measuring deflections at various points. Openings were cut in the concrete to the depth of the steel bars at [illegible] points along the lower flange of the girders and in the floor in order to locate gage holes for use in measuring deformations in the steel. For measuring concrete deformations at the upper surface of the girders, steel plugs were set in small holes drilled in the concrete. The outsides of the girders were white-washed in order to facilitate the mapping of cracks.

Method of Loading. It was desired to apply the load in a manner that would approximate as nearly as possible the conditions of a uniformly distributed load on the bridge floor. The principal loading material consisted of crushed limestone from the prison crusher nearby. In order to prevent arching of the loading material due to the deflection of the girders and floor, it was deemed advisable to build a series of separate bins for the crushed stone. The bins were eight in number, 7 ft. square inside, placed four on each side of the center line of the bridge. They were built up of 2x6-in. lumber, laid flatwise and spiked together at the corners, the edges of the outer bins being 2½ ft. inside the outer faces of the abutments. The bins were first built up to a height of about 5 ft. over the entire floor. A runway was then constructed over the tops of the bins and the crushed stone dumped from wheelbarrows. After this portion of the load had been placed, the bins were carried up a few feet at a time, and the remaining portion of the stone load was placed by means of a horse-power derrick, which was erected at the side of the bridge. Each wagonload of stone was weighed on a platform scale before it was dumped at the bridge site, so that at any time the exact weight of stone on the bridge could readily be determined.

The stone bins were finally filled to a height of 21 ft., which was about the maximum height to which it seemed feasible to build them. The total load on the bridge at this time was 418 tons. Later, about 144 tons of pig iron were placed on top of the girders without throwing any additional load on the floor.

Progress of Loading.—A detailed schedule of the loading and unloading is given in Table I.

TABLE I.—PROGRESS OF LOADING THE TEST BRIDGE.

Date—	Remarks.
July 26, 1911	Began building stone bins on bridge floor. Built brick piers for deflection measurements. Took series of temperature readings.
" 27	Continued building bins; took temperature readings, initial deflection readings and initial readings of steel and concrete deformations.
" 28 to 30	Load of 6 tons on bridge, consisting of lumber in the stone bins. Took frequent series of temperature, deflection and deformation readings.
" 31	Began loading stone. Total load on bridge at 4:30 p. m., 89 tons.
August 1	Loading stone. Load at 4:30 p. m., 122 tons.
" 2	Built stone bins in forenoon. Load on bridge at noon, 128 tons. Began loading stone 2:30 p. m. Load at 4:30 p. m., 166 tons.
" 3	Began loading stone at 8 a. m. Load at 11 a. m., 219 tons. Built bins in afternoon; load at 5 p. m., 225 tons.
" 4	Began loading stone at 9 a. m. Load at 4:30 p. m., 280 tons.
" 5	Built bins; load at 4 p. m., 283 tons.
" 6 to 7	Load on bridge remained at 283 tons.
" 8	Began loading stone at 8 p. m. Load at 5 p. m., 335 tons.
" 9	Loading stone and building bins; load at 5 p. m., 369 tons.
" 10	Loading stone and building bins; load at 5 p. m., 418 tons.
" 11	Took full set of readings, plotted cracks and took photographs. Load remained at 418 tons.
" 12 to 25	Load, 418 tons. Took set of readings on August 25.
" 30	Began loading pig iron; 12 tons placed on each girder.
" 31	Loading pig iron on girders; at 2 p. m., 44 tons had been placed on each girder. No further load was put on east girder. Continued loading pig iron on west girder. At 4 p. m., 80 tons pig iron on west girder.
September 1	Loading pig iron on west girder. Load at 11:30 a. m., 100 tons. This was the maximum load applied to the test bridge—100 tons of pig iron on the west and 44 tons on the east girder, in addition to the 418 tons of stone on the bridge floor.
" 2 to 4	Load remained on the bridge as above.
" 5	Began removing pig iron load.
" 6	All pig iron removed from the girders.
" 7	Took full set of observations under stone load of 418 tons.
Sept. 8, 1911, to Aug. 12, 1912	Load of 418 tons remained on bridge. Set of readings taken Dec. 29, 1911.
August 12, 1912	Took set of readings under load of 418 tons.
" 13	Began unloading stone. Load at 4:30 p. m., 330 tons.
" 14	Unloading stone; load at 11 a. m., 274 tons. Load at 4:30 p. m., 205 tons.
" 15	Unloading stone; load at 11:30 a. m., 160 tons. Load at 5 p. m., 110 tons.
" 16	Unloading stone; load at 11 a. m., 100 tons. Load at 5 p. m., 70 tons.
" 17	Unloading stone; load at 3 p. m., 30 tons.
August 17 to September 3	Load remained as above.
September 3, 1912	Took deflection readings.
" 4	Remainder of load removed.
" 5	Took final observations.

Three or four days were spent in preparing the bridges for loading, and in making deflection and deformation measurements, in order to study the effect of diurnal temperature changes. The observations of temperature, deflection of girders and floor, and the deformations of concrete and steel at the various points, were taken at frequent intervals during the period of applying the load, and while the load was in place. In general, a complete set of observations was taken at an early hour each day, before the morning had an appreciable effect, and another set after sunset. It was hoped that in this way the effect of the direct sun heat on the bridge and the measuring instruments would be less troublesome. Observations of temperature and deflections were taken at more frequent intervals during the day.

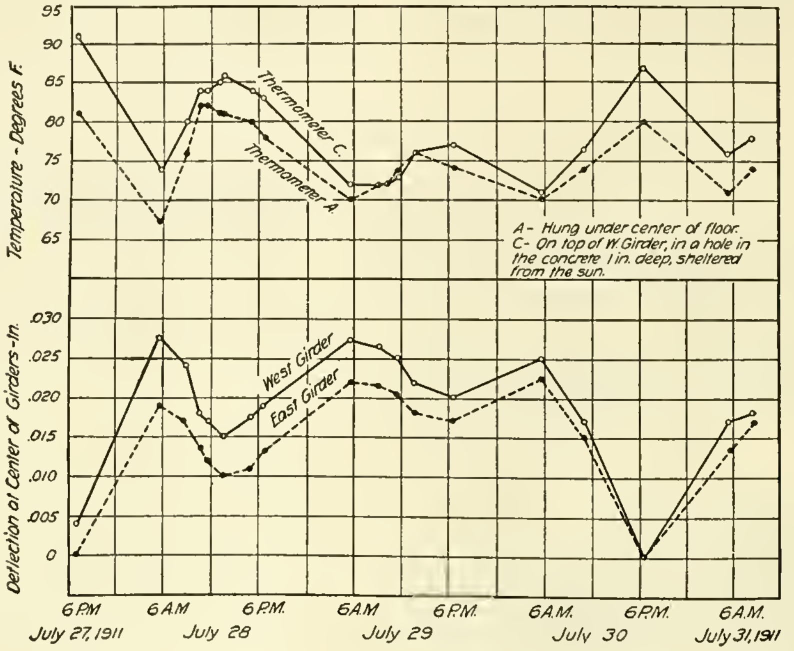


Fig. 3. Diagrams Showing Effect of Temperature on Deflection of Girders.

On Aug. 10, 1911, a total stone load of 418 tons had been applied to the floor. The load remained unchanged until Aug. 30, when the placing of pig iron on the tops of the girders was begun. The pig iron load was distributed over the middle of 30 ft. of the 40-ft. span. At first a load of 44 tons was placed on each girder. It was then seen that the amount of iron available, if placed equally on both girders, would not produce as high stresses as had been expected; therefore, all the remaining pig iron was placed on the west girder. On Sept. 1, a load of 100 tons of pig iron had been placed on the west girder, and 44 tons on the east girder, in addition to the 418 tons of stone load.

It was the expectation that in this test the bridge would be loaded to failure, but the loading material available and the facilities at hand did not permit. The maximum load remained undisturbed from Sept. 1 to 5, 1911. On Sept. 6, the pig iron load had been entirely removed, but the stone load of 418 tons was allowed to remain in place until Aug. 13, 1912. One complete set of observations were taken on Dec. 29, 1911, and on Aug. 13, 1912, the removal of the stone and bins was begun. Observations of the deformations in the concrete and steel of the girders and deflections of the girders were taken during the unloading of the stone, and after the removal of the load. Final observations were taken on Sept. 5, 1912. It will be seen that the bridge carried an applied load equivalent to 318 tons per girder for four days and a floor load of 418 tons for over a year.

Measurement of Deflections.—All measurements of deflection in the girders and floor were taken from the brick piers. The deflection measurements were taken by means of an Ames gage attached to a screw micrometer. As an illustration of the sensitiveness of this instrument in indicating changes of deflection, and also to show the action of the bridge, it may be of interest to note that with the full stone load on the bridge—418 tons, in addition to the weight of the bridge—an appreciable movement of the pointer of the deflection instrument was produced by a man walking along the top of one of the girders.

Temperature Observations.—In the preliminary tests mentioned above, the changes in the position of the girders due to temperature variations made the interpretation of the observations difficult, and showed the necessity of determining in advance the effect of temperature. In the final test, three thermometers were used and the readings recorded at frequent intervals. Three or four days before the application of load was begun, frequent observations of temperature and deflections were made for the purpose of determining the effect of the diurnal changes in temperature on the position of the bridge.

Measurement of Steel and Concrete Deformations.—Tensile deformations of the steel were measured at the mid-points of the girders and at points about 5 ft. each side of the middle, and at two points near the center of the bridge floor. The measurements at the center of the girders were taken on the three bars of the bottom layer, and the other girder measurements on the middle bar only. The deformations of the floor steel were measured on two ⅞-in. bars about 12 ins. apart, near the middle of the span. Deformations were also measured at points near the middle of two ¾-in. vertical stirrups, about 6 ft. from the north end of the west girder. Concrete deformations were measured at two points at the middle of the top of each of the girders and at single points about 5 ft. each side of the middle. All deformation measurements were made by means of a modified form of the Berry strain gage.

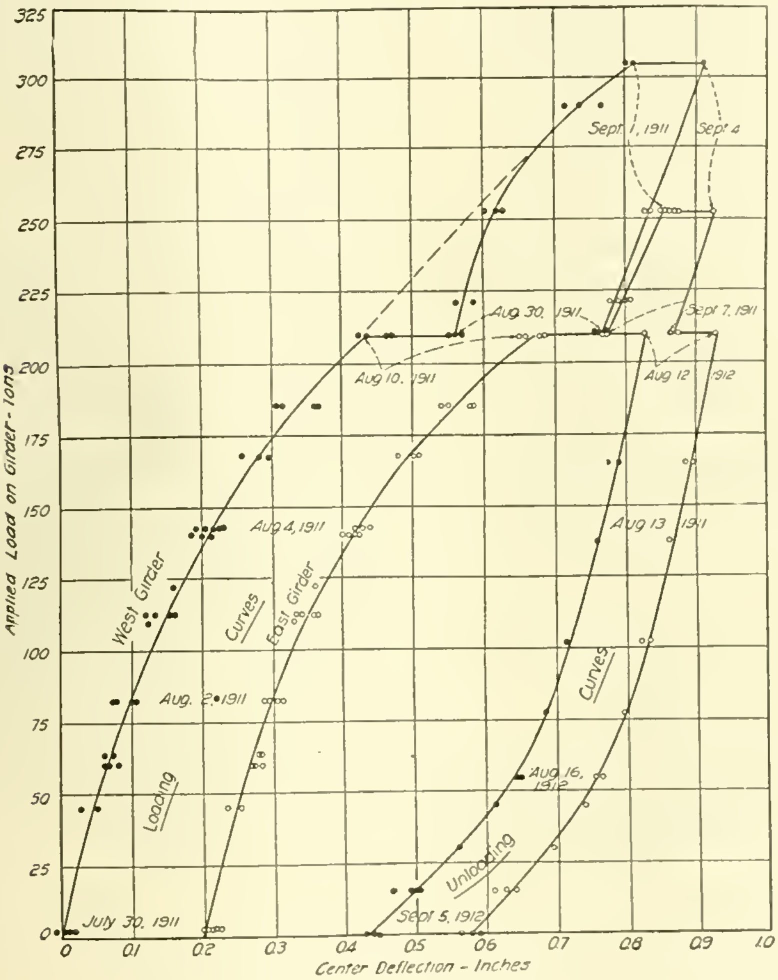


Fig. 4. Load-Deflection Curves for Girders.

RESULTS OF TEST

Effect of Temperature on the Deflection of the Girders.—In Fig. 3, the relation of temperature to the position of the bridge is shown for the four days preceding the commencement of loading. The atmospheric temperature ranged from 67° to 83° F. The thermometer "C" on top of the west girder indicated the effect of heating the concrete. This effect is most noticeable during the afternoons after the girders have been exposed to the sun for a few hours. It is noteworthy that the west girder showed more change in deflection than the east, but the east girder was protected from the sun by the prison wall until about the middle of the forenoon. The thermometer on top of the girder did not indicate the highest temperature to which the concrete was exposed, but it probably indicated the degree of heat which may be considered as effective in producing changes in the position of the girders.

The deflection of the girder is seen to vary directly with the change in temperature at the top of the girder, rather than with the atmospheric temperature. The maximum change in deflection due to this cause with the bridge unloaded was 0.027 in. The reason for the greater influence of the temperature change at the top of the girder is readily found in the fact that the concrete at this point is heated to a higher degree than elsewhere and the longitudinal expansion of the concrete due to this cause has the effect of lengthening the top chord of the girder, and, since the lower portion is not thus elongated, the girder must assume a curved form, and thus raise its center above its former position. As indicated in Fig. 3, the change in deflection at the centers of the girders amounts to about 0.001 in. per degree.

These considerations indicate that in any set of observations in which careful measurements of deflection are taken on a structure of this kind, the results must be interpreted in conjunction with the temperature changes which are encountered. In subsequent figures in which the deflections of the girders have been plotted, no correction for temperature has been made, but the group of points which appears for each load shows the maximum variations due to temperature during the time covered by the measurements under the corresponding load. When a mean line is constructed, the error due to this cause is negligible.

Deflection of Girders under Load.—In Fig. 4 the center deflections for both girders have been plotted for the loading and unloading of the bridge. The approximate dates corresponding to some of the points are indicated in the figure. The action of the girders is quite similar to that which has been observed for reinforced concrete beams under load. Up to an applied load of 209 tons on each girder the curves have the characteristic parabolic form.

During the interval from Aug. 10 to 30, 1911, with the load at 209 tons, the deflection changed from about 0.42 to 0.56 in. After the placing of the pig iron load was begun on Aug. 30, the change in deflection was comparatively slight for the first portion of the load. Later, the curve follows a path which might have been predicted for it had the application of the load not been interrupted. During the four days which the west girder remained under the full applied load of 309 tons, the center deflection changed from 0.81 to 0.91 in. No measurements of deflection were taken during the unloading of the pig iron. After the removal of the pig iron the west girder deflection was about 0.30 in. greater than it was immediately after the stone load was applied. During the interval of about eleven months after the removal of the pig iron, with the stone load in place, the change in deflection of both girders was about the same as that which occurred during the 20 days following the placing of the stone load. This, however, is as might have been expected, since the bridge had been subjected to a much higher load in the meantime. After unloading, the girders showed a permanent deflection of about 0.45 in.

In Fig. 5 several curves have been drawn which show the deflection of the west girder at the various points where measurements were taken, for loads varying from 45 to 309 tons. These lines may be said to represent the elastic curves for the girder. The remarkable symmetry of the curves will be noted. By projecting the curves beyond the last observation points to the abutments, it is seen that there was a slight settlement of the ends of the girders as the load increased. This was probably the result of the concentration of the load at the outer face of the abutments due to the deflection and tilting of the girders. However, this is a small amount and no correction has been made for it in plotting the curves.

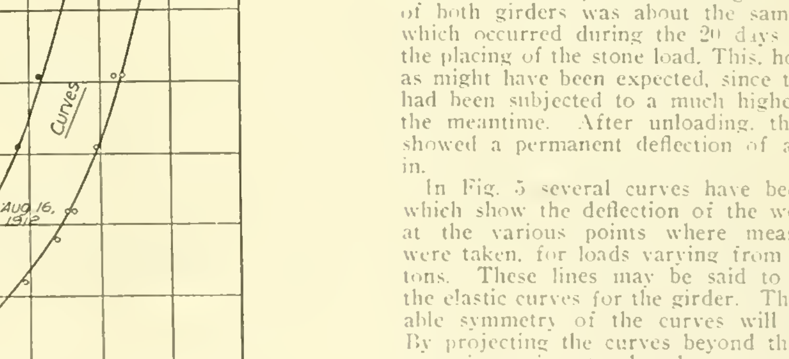


Fig. 5. Deflection Curves for West Girder.

Deflection of Floor Slab under Load.—In Fig. 6 the deflections of the center of the floor slab have been plotted in terms of the stone load, considered as uniformly distributed over the floor. The maximum load placed on the floor amounted to about 1150 lbs. per square foot. The deflections given for the floor have been reduced in such a way that they represent the true deflection of the floor slab with reference to the girders and not the total depression of the floor at this point. The maximum deflection of the floor was about 0.96 in. Upon removal of the load, the floor slab showed a residual deflection of 0.37 in.

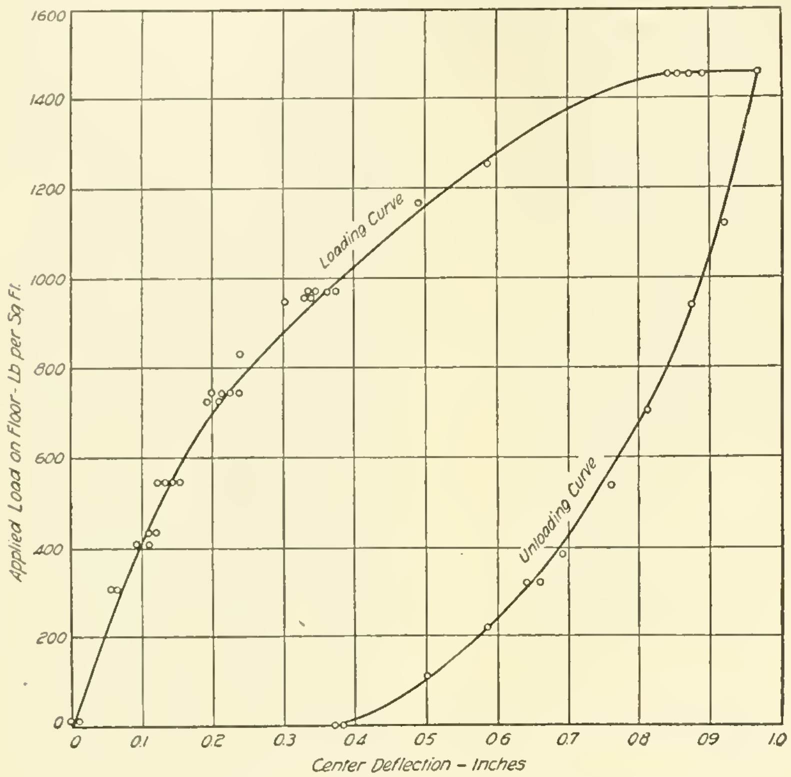


Fig. 6. Load-Deflection Curves for Floor Slab of Bridge.

Effect of Temperature on Deformation Measurements.—It should be borne in mind that in attempting to make accurate measurements of deformations in the concrete and steel of a structure of this kind, the difficulties are greatly increased over those encountered in the usual indoor laboratory test. Variations in temperature and other changes in weather conditions affect the structure as well as the instrument used, and under the very best conditions wide variations may be expected in observations of this kind. This is especially true of the concrete deformations which were taken on top of the girders, since the readings were affected by the local heating of the girders, and by the direct rays of the sun and the radiated heat on the instrument. It was found that the deformation measurements taken at hours when the direct rays of the sun fell upon the instrument were not very reliable.

The observations made before loading was begun did not cover a sufficient range to indicate the exact effect of temperature variations on the concrete and steel deformations. However, the range of the points for certain loads on the load-deformation diagrams for the girders show the extreme variations which were found.

There are so many variables which affect a measurement of this kind that it is impossible in this case to differentiate between the errors due to instrument adjustment, the personal equation of the observer, and the actual variations in the deformations due to stress or temperature. If it were not true that some engineers entertain mistaken ideas on this point, it would be considered superfluous to say that the changes in the position of the girders in a bridge of this kind, and changes in deformation due to temperature, have no influence on the stresses developed in the concrete and steel, as long as the ends of the girders are not fixed. It should be emphasized that it is change in deformation that we are now dealing with and as long as changes in temperature affect the results this does not necessarily mean change in stress. In fact, in some cases the change of deformation due to change of stress seems to have been entirely masked by the change of deformation due to temperature.

The deflection readings were not appreciably affected by changes in temperature, since the quantities dealt with were larger, and there was little chance for variation in the adjustment of the instrument.

Deformations in Girders.—In this discussion only the deformations taken at the centers of the girders will be considered.

Figure 7 shows the unit deformations in the steel at the centers of both girders, plotted with respect to the applied load in tons on the girders. Approximate dates are shown on the diagram. It will be noted that these load-deformation curves exhibit about the same characteristics as were described for the load-deflection curves. The observed stress in the steel at the center of the girder under an applied load of 102 tons per girder averaged about 11,700 lbs. per square inch. The deformations were measured on the lowest layer of bars, and they would be expected to represent the highest stresses developed in the girder steel. Under the full applied load of 309 tons, the west girder showed a stress of 23,400 lbs. per square inch, which apparently was increased to about 25,500 lbs. per square inch during the time this load remained in place. After the removal of the load, the west girder showed a residual stress of about 13,000 lbs. per square inch. It seems probable that if subsequent observations had been taken, this residual stress would have been found to be greatly reduced after a period of rest. The action of the two girders was very similar, except that there was a difference in the applied load on the girders.

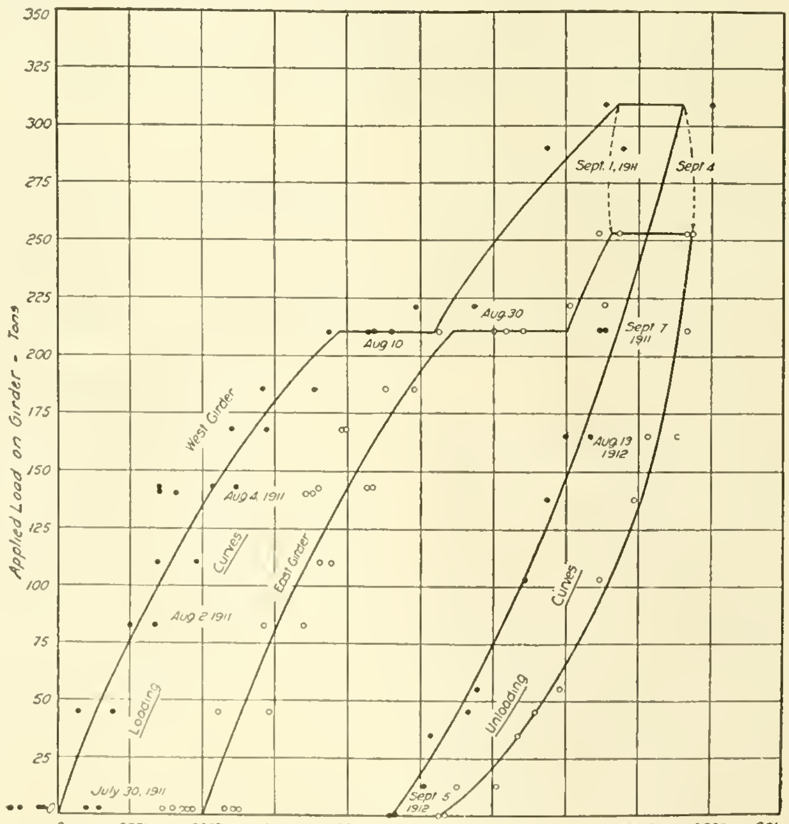


Fig. 7. Load-Deflection Curves for Steel in Girders.

In Fig. 8 the deformations of the concrete at the top of the west girder have been plotted. The curve is shown only to an applied load of 250 tons on the girder. The unit deformation for the maximum load derived from producing the curve is about 0.0011, which is probably 80 per cent of what might be expected for the ultimate deformation in concrete of this quality and age, and since this deformation in the concrete is due to applied load only, it represents a stress which must be near the ultimate compressive strength of the concrete.

Position of Neutral Axis.—Before making computations of stresses in any reinforced concrete member subjected to flexure, it is necessary to know the position of the neutral axis, or to make some assumption as to its location. It is evident that in a structure of such irregular section as the test bridge under consideration, the location of the neutral axis from theoretical considerations will be somewhat problematical, and that its position will vary widely with the assumptions made as to the properties of the concrete and the distribution of the stresses between the concrete and steel. The calculated depths of the neutral axis below the tops of the girders for various conditions are as follows:

Girder only, considered as plain concrete beam	38 ins.
Girder only, steel taking all tensile stress.	26 ins.
Girder only, both steel and concrete taking tension	31 ins.

Bridge as a whole, both concrete and steel in floor and girders taking tensile stress. 45 ins.

In computing the foregoing values, the modulus of elasticity of concrete was taken at 3,000,000 lbs. per square inch. However, a considerable variation in this modulus makes only a slight change in the computed depth of the neutral axis.

The deformations measured in the concrete at the tops of the girders and in the lower layer of girder bars give definite information as to the neutral axis position during the loading of the bridge. Table II shows the position of the neutral axis as determined from the deformations in the diagrams in Figs. 7 and 8, for applied loads on the west girder, varying from 50 to 300 tons. The values from Table II have been plotted in Fig. 9; they indicate a steady rise of the neutral axis from about 50 ins. under dead load only to 40 ins. under an applied load of 309 tons, with a mean value of about 44 ins. It was seen above that a depth of 45 ins. was computed for the condition that both concrete and steel in the entire section of girders and floor are considered as effective in taking the longitudinal tensile stress. The close agreement between the observed position and the computed position for the bridge as a whole indicates that the girders did not act as independent reinforced concrete beams in the manner usually assumed, but that the concrete in the girders and floor is effective in taking tensile stresses. The gradual rise of the neutral axis as the load was increased is in accord with other tests of reinforced concrete beams. This rise is probably due to the opening of tensile cracks which extend up a short distance in the concrete, to a slight slipping of the bars in the concrete, and to the increasing deformation at the tops of the girders as the loading progressed. In comparing the depth in inches with the percentages of the effective depth, it should be remembered that the steel deformations were measured on the lowest layer of bars—about 70 ins. below the top—while the effective depth of the girder was 66 ins.

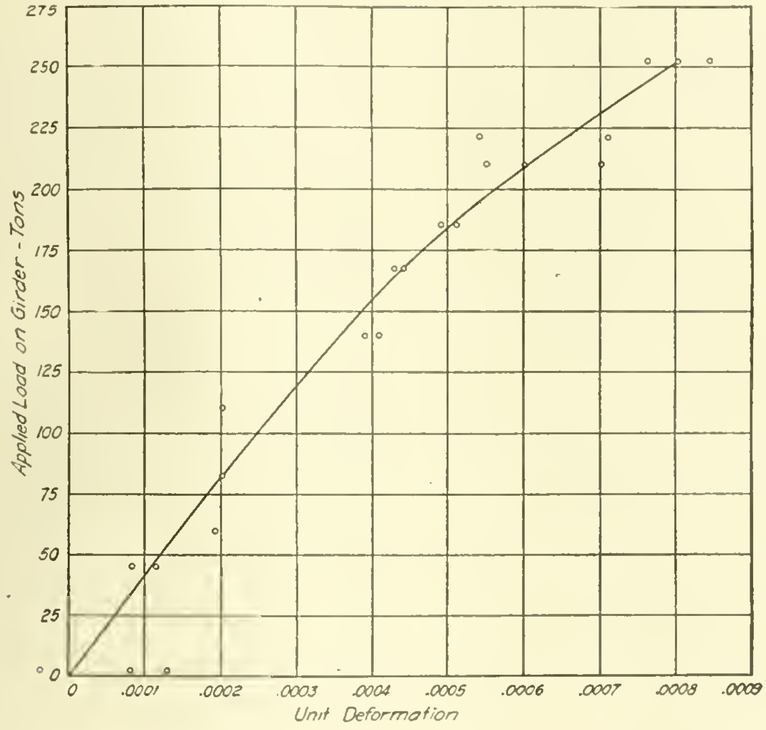


Fig. 8. Load-Deformation Curve for Concrete at Top of Girder.

TABLE II.—NEUTRAL AXIS POSITION, DETERMINED FROM MEASURED DEFORMATION IN THE GIRDERS.

Applied load on girder, tons.	Unit deformation: Concrete at top of girder.	Unit deformation: Steel at bottom of girder.	Neutral axis position: Distance below top surface of girder, ins.	Neutral axis position: Percentage of effective depth.
0	....	....	50*	76*
50	0.00012	0.000055	48	73
100	0.00025	0.00013	46	70
150	0.00039	0.00023	44	67
200	0.00056	0.00035	43	65
250	0.00080	0.00054	42	64
300	0.00104*	0.00074	41	62
309	0.00110*	0.00083	40	61

*Estimated.

Computed Stresses in Girders. For purposes of computation the load will be considered as uniformly distributed over the full span of 40 ft., although the pig iron load was distributed over a length of 30 ft. and the stone load over a length of 35 ft. This gives a bending moment about 12 per cent less than that resulting from taking the loads as they existed, but this error is offset by the consideration that a portion of the stone load on the floor was carried directly to the abutments without producing any stress in the girders.

The bending moment in one girder due to the maximum test load, considering the girder as a simple beam uniformly loaded, is

$$M_b = \frac{618{,}000 \times 40 \times 12}{8} = 37{,}100{,}000 \text{ in.-lbs.}$$

The resisting moment of the girder, considering the steel to take all the tensile stresses, is

$$M_r = Af_sd'.$$

Equating bending moment to resisting moment,

$$f_s = \frac{M_b}{Ad'}$$

where A = area of longitudinal steel in girders, 15.5 sq. ins.;
f_s = computed tensile stress in steel, and
d' = arm of resisting moment.

Taking the depth of the neutral axis as 40 ins., as found for the maximum load from the deformation measurements (see Table II), the centroid of the compressive stresses is found to be 15 ins below the top of the girder, or at about what we may term the lower edge of the coping. This gives d' = 51 ins. Substituting these values, and solving for the stress in the steel, f_s = 47,000 lbs. per square inch, if we consider all tensile stress taken by the steel of the girders. If the dead load on the girders is included, the computed steel stress becomes about 62,000 lbs. per square inch.

The maximum steel stress determined from the deformation measurements was 25,500 lbs. per square inch, which means that the measured stress in the steel accounts for only 25/47 of the total tensile stress resulting from the applied load on the bridge. If the deformation measurements are to be depended upon, 22/47 of the total tensile stress must be carried by the steel in the floor and by the concrete in the floor and girders. If we consider the longitudinal floor steel as carrying its part of the stress in the same way as the girder steel, the average computed steel stress is reduced to about 40,000 lbs. per square inch, which indicates that about 36 per cent of the total tensile stress must be carried by the concrete of the floor and girders. Taking the moment of inertia of the entire concrete section about the neutral axis as determined from the deformation measurements—40 ins. below the top of the girders—and making the necessary computations, we find that a maximum tensile stress in the concrete at the lower surface of the bridge of about 300 lbs. per square inch will be sufficient to account for all tensile stress not taken by the steel. This assumes, of course, that the steel is taking all the dead load stress, which is probably true; since at the time the centers were struck, the tensile strength of the concrete was comparatively low, and the concrete had ample time during the subsequent period of hardening to thoroughly adjust itself and come to an unstressed condition before the application of the final load. It is probable that concrete of this age would carry a tensile stress much higher than 300 lbs. per square inch without failure.

The computed compressive stress in the girder from live load only is about 2200 lbs. per square inch on the assumption that the bridge as a whole takes the stress, with the neutral axis at a depth of 40 ins. For combined dead and applied load, this stress amounts to about 3000 lbs. per square inch. The maximum measured unit deformation in the concrete at a load of 250 tons on the girder was about 0.0008, which indicates a deformation of about 0.0011 at the maximum load of 309 tons. Assuming an initial modulus of elasticity of 3,000,000 lbs. per square inch for the concrete, this deformation under maximum applied load only indicates a compressive stress of about 2000 lbs. per square inch. It may not be expected that the computed and the measured stresses in the concrete will show a close agreement, since the concrete deformation measurements were less satisfactory than those for the steel, and the strength and elastic properties of the concrete are uncertain.

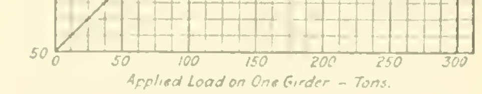


Fig. 9. Observed Neutral Axis Positions of Cross Section of Bridge.

It is recognized that there are discrepancies in the above analysis of the distribution of tensile stresses, for it would be difficult to account for the resulting deformations in the steel and the adjacent concrete if the stresses were distributed as indicated. However, the low position of the neutral axis, the low stresses measured in the girder steel, the minuteness of the tensile cracks in the girders, the rigidity of the connection of the floor to the girders, and the general behavior of the bridge under these unusual loads, will admit of no other explanation.

Stresses in the Vertical Stirrups.—Vertical stirrups were provided to take the shearing stresses developed in the girders. Deformations were measured on two ¾-in. stirrup bars at about 6 ft. from the north end of the west girder. These measurements were not begun until a load of 140 tons had been placed on the girder. From this point to the maximum load the measured deformations indicate an increase in stress of about 12,000 lbs. per square inch. The close spacing of the diagonal cracks in this girder indicated that the web stresses were well distributed, and that the stress in the vertical stirrups was not high.

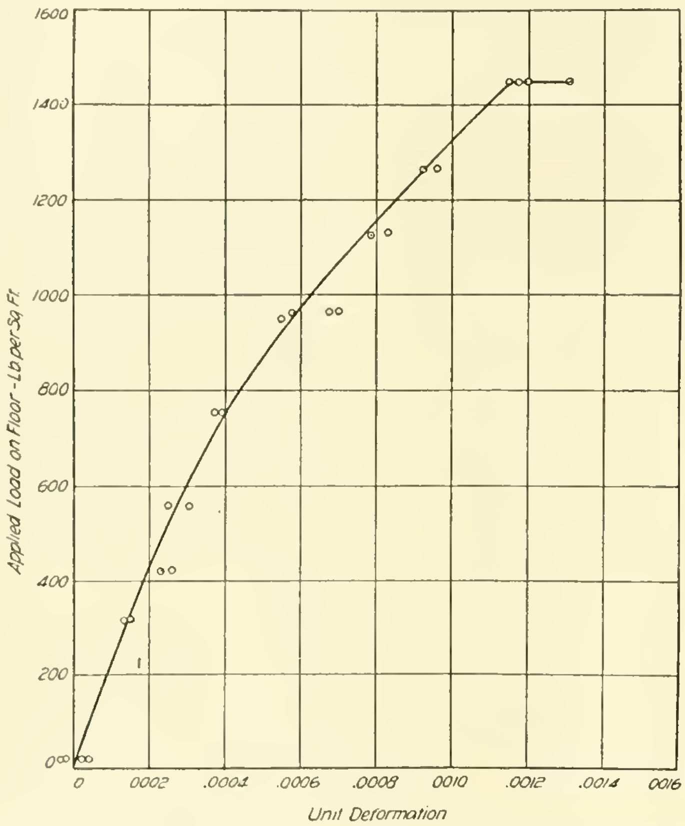


Fig. 10. Load-Deformation Curve for Floor Steel of Bridge.

Deformation and Stress in the Floor Reinforcement.—Measurements were taken at the middle of two ⅞-in. floor bars near the middle of the floor span. These measurements were subject to much less variation than was found in the girders, probably due to more constant temperature at this point. The values have been plotted in Fig. 10. The curve exhibits the usual characteristics for steel deformations in reinforced concrete beams. At the time of the application of the full stone load on the floor, these bars showed a measured stress of about 36,000 lbs. per square inch, which was increased to about 40,000 lbs. per square inch before the stone load was removed. It was not practicable to make observations on these bars during the removal of the stone load. It was impossible to take deformation measurements in the concrete of the floor.

In computing the stresses in the floor-slab reinforcement, it was considered that the load was carried laterally by the system of ⅞-in. twisted square bars, 4¾ ins. apart, to the lower flanges of the girders. For purposes of comparing the computed stress in the reinforcing bars with the stresses found from the observed deformation at the middle of the floor slab, it is sufficiently accurate to consider that the entire stone load of 418 tons was uniformly distributed over the area covered by the bins. It is appreciated that a portion of this load near the ends of the bridge was carried directly to the abutments by the floor slab and did not produce stress in either the main system of floor reinforcement or the girders, but since we are concerned only with what occurs at the middle of the bridge span, it is clear that the above method is proper. For the full applied load of 418 tons this gives a uniformly distributed load of 1450 lbs. per square foot on the floor.

If we consider that the floor slab carried its load as a partially fixed beam, it will be of interest to determine the elements of a beam which would develop the same stress in the reinforcing steel as that measured in the floor steel in the test. Using a uniformly distributed load of 1450 lbs. per square foot, and an effective depth of 10.7 ins., we find that a simple beam of 15.5-ft. span would develop a stress equal to that measured in the floor steel. In other words, we may assume that the points of inflection of the floor slab are 9 ins. from the wheel guard, and that between these points the floor acts as a simple beam. If the floor slab was fully fixed by the girders, the points of inflection would be about 0.21 of the span, or about 3½ ft. from the wheel guard. These considerations indicate that although the floor is rigidly attached to the girders, its ends have only a slight degree of fixedness, on account of the freedom with which the girders may deflect laterally and tilt inward at the top as the floor deflects under the load. It should be borne in mind that the stresses discussed above are in addition to the dead load stress which has not been considered. The computed dead load stress in the floor-slab steel amounts to about 4,000 lbs. per square inch. The numerous cracks which were present and the measured deflections indicate that the floor steel was stressed much higher than the girder bars.

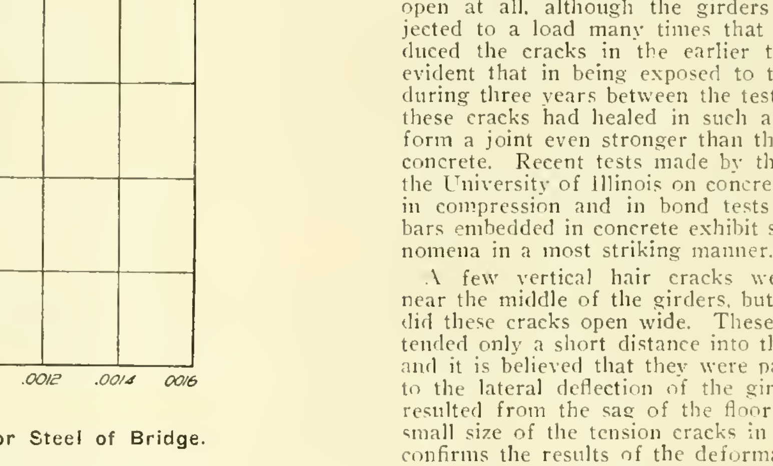


Fig. 11. Diagram Showing Cracks in Slab of Bridge.

Cracks in Girders and Floor.—Before beginning the last test the cracks in the girders produced by the preliminary tests were carefully mapped and photographed. These cracks could be traced on each end of both girders, inside and outside. Photographs were taken which show similar cracks at the other ends of the girders. It was found in many instances during the application of load that diagonal cracks which had been previously marked had opened again, but at a much higher load than was placed on the bridge in the earlier tests. In some instances new cracks were formed parallel to and near the old ones, and in some cases the cracks which were produced by the earlier tests did not reopen at all, although the girders were subjected to a load many times that which produced the cracks in the earlier tests. It is evident that in being exposed to the weather during three years between the tests, many of these cracks had healed in such a way as to form a joint even stronger than the unbroken concrete. Recent tests made by the writer at the University of Illinois on concrete cylinders in compression and in bond tests with plain bars embedded in concrete exhibit similar phenomena in a most striking manner.

A few vertical hair cracks were formed near the middle of the girders, but in no case did these cracks open wide. These cracks extended only a short distance into the concrete, and it is believed that they were partially due to the lateral deflection of the girders which resulted from the sag of the floor slab. The small size of the tension cracks in the girders confirms the results of the deformation measurements in the longitudinal steel, in showing that at no time was the steel highly stressed.

Cracks in the floor slab were much more prominent and much more numerous than in the girders. These cracks were well distributed and extended parallel with the girders throughout the middle half of the bridge span. The fine tension cracks which opened in the lower portion of the girders extended a short distance along the lower surface of the floor slab. Fig. 11 shows the approximate position of the principal cracks on the underside of the floor after the full load had been applied. The figure has been drawn as seen from below. The heavy dotted lines across the corners indicate the position of the cracks on the top of the floor. The top cracks were nearly at right angles to those in the bottom face of the floor slab. They were also much more prominent than those at the bottom of the floor, and extended well into the floor concrete and up to the middle of the inside of the girders, and in two instances they joined diagonal cracks which had been formed at earlier stages of the test. It was found that the deflection of the bridge had caused the ends of the floor to be lifted from the abutments. It is plain that the peculiar condition under which the floor slab is loaded in a bridge of this type, may be expected to produce just the system of cracks which were observed in this test.

Behavior of the Abutments.—As stated previously, the deflection measurements near the ends of the girders indicated that there was some settlement of the ends of the girders under the load. This can be accounted for by the consideration that the deflection and the inward tilting of the girders produced a highly concentrated bearing stress on the concrete near the outer faces of the abutments which probably resulted in crushing the concrete at these points to the extent shown. The effect of the deflection and tilting of the girders is also shown by the horizontal cracks which formed across the tops of the wing walls and across the ends of the girders.

SUMMARY.

In arriving at a final estimate of the value of the information derived from this test, it should be borne in mind that at the time of the final loading the bridge was 3½ years old. However, the influence of the age of the concrete was somewhat offset by the unfavorable conditions under which it was built. This bridge of 40-ft. span carried a load for four days which was equivalent to 309 tons per girder, in addition to the weight of the bridge, and the floor slab carried an applied load of 418 tons for over a year, without indication of serious distress. These loads amount to about 9 to 13 times the maximum load which a bridge of this kind would ever be called upon to carry in service. The remarkable load carried by the floor and girders may suggest that the bridge was over-reinforced, or that the design was heavier than necessary; but it should be remembered that the strength of the concrete and the stiffness of the bridge as a whole were very much increased over what we would find in the structure during the first few months after it was put into service.

The behavior of the girders as indicated by the opening of the diagonal cracks in the webs during the preliminary tests, as compared with the final loading, is an evidence of the increased strength and stiffness of the concrete. This calls attention to one of the most valuable properties of reinforced concrete for bridge construction. In the case of a steel structure, great care must be used to prevent the deterioration from beginning at once; while bridges of concrete increase in strength and stiffness with the lapse of time, and are all the better structures for having been exposed to the weather for several years.

The maximum stress in the longitudinal girder steel, as determined from the deformation measurements, was about 25,000 lbs. per square inch. The stress computed for the same load, under the assumption that the steel of the girders takes all the tensile stress, is 47,000 lbs. per square inch. It will be appreciated that these girders are of irregular form and unusual size; they are not comparable in size and form with other reinforced concrete beams on which tests have been reported. The ordinary assumptions may not apply to a girder of this form and size. The unknown values of the strength and stiffness of the concrete in this bridge is an additional element of uncertainty. The wide variations between the computed stress in the longitudinal steel of the girders and the measured stress is probably due to the fact that the girders do not act as independent beams, but the entire concrete section and the steel of the floor are also effective in taking the longutudinal stress. The manner in which these stresses were distributed between the girder steel and the floor was not determined; but it is apparent from the general behavoir of the bridge, the low position of the neutral axis, and the minuteness of the tension cracks in the girders, that the steel stress was actually comparatively low, and that both concrete and steel of the floor were effective in taking this stress. It is evident that the influence of the floor is much greater at this age than it would have been during, say, the first few months after opening the bridge to traffic. The measurements also indicated that while the compressive stress in the tops of the girders are very high, it was still within the ultimate strength of the concrete.

It was impracticable to compute the diagonal shearing stress developed in the webs of the girders, on account of the irregular section of the girders and the unknown action of the floor, but it is evident that while this stress was very high, the girders were not seriously affected in carrying it. The numerous diagonal cracks in the girders, and the low stresses measured in the vertical stirrups, show that the vertical stirrups provided ample reinforcement against the stresses developed.

The inward tilting of the girders in response to the deflection of the floor shows the remarkable rigidity and strength of the connection of the suspended floor slab. The detail of the connection of the floor slab to the girders may be considered a distinctive feature of this design for a highway bridge, and its action under an unusual test load gives added confidence to this method of construction.

The load of 418 tons carried by the floor for over a year is equivalent to a uniformly distributed live load of about 1450 lbs. per square foot. The measured stress in the floor steel under this load was about 34,000 lbs. per square inch, which corresponds to the computed stress, if we consider the effective span of the floor slab to be somewhat shortened by a certain degree of end-fixedness, resulting from the method of attaching it to the girders.

Water Works

Desirability of Making More Intelligent Use of Underground Space in Locating Street Mains and Conduits of All Kinds.

The laying and maintaining of street mains, particularly in the congested districts of large cities, constitutes one of the serious problems now confronting the distribution departments of water and gas companies, sewer departments and all other interests that are compelled to use the streets for the laying of mains and conduits. Formerly even in large cities this condition did not exist, for the water main was laid 6 ft. from one curb, the gas main 6 ft. from the other curb, and the street car tracks occupied the center of a few streets The development of public utilities of all kinds has been so great in recent years that all is now confusion amongst the various underground pipe lines, conduits and appurtenances lying beneath the street surfaces in the business districts of the larger cities. Conditions are especially bad in Boston, and Mr. J. A. Gould in a paper before the American Gas Institute recited the conditions existing in that city with a view to indicating the desirability of making more intelligent use of underground space for the purposes here under consideration in cities which still have the opportunity to apply modern methods in locating pipes under streets. Mr. Gould writes particularly from the point of view of the gas man. Something of this point of view should be understood by public works officials, especially since in most states the privately owned gas main must give the right of way to almost everything in the shape of publicly owned pipes or conduits. The portions of the paper which follow are of interest to all public service departments or corporations:

The Boston sewer department now has a double system of sewers—one for house drainage, the other for surface drainage. The water department has its ordinary distributing mains, a duplicate system of high-pressure mains for fire service, and reinforcing feed mains. The electric business has developed rapidly until there are now nearly a score of different varieties of wires buried in our streets, among which are found the following companies:

Street railway company's feed and return lines, the electric illuminating, the New England Telephone and Telegraph, American Telephone and Telegraph, Western Union Telegraph, Postal Telegraph, Telepost, fire alarm service, police signal service, Boston District Messenger, Automatic Fire Alarm, Electric Protective, Holmes Electric Protective, Mutual District Messenger, Stock Quotation Telegraph and United Telegram.

Several of these larger interests, like the street railway, the lighting, the telephone and telegraph companies, have separate conduits, while nine of the companies using low voltage combine as the Low-Tension Wire Association, and their wires are laid in a common conduit These electric conduits may be of small cross section, but as the manholes are built on too liberal a scale, they encroach upon the neighboring pipes and make serious complications. In addition to the electric conduits we have the pipes of the heating company followed by those of the freezing companies. The pneumatic tubes for the mail service and parcel delivery are also troublesome neighbors, because exacting in their requirements as to line and grade.

The requirements of transportation have outgrown the capacity of the surface lines and require the elevated system with its large underground piers, or subways which tear up all existing gas pipes and most of the other structures. The city catchbasins, which occupy a portion of the sidewalk and about 3 ft. of the roadway, still further reduce the available space; and in the business districts the practice of occupying the space under the sidewalks for vaults still further reduces the area available for pipes and conduits.

One very serious condition that confronts gas companies that are not municipal plants is they have no permanent rights in their locations that the city or state is bound to respect when the latter wants to do any work with which our pipes conflict. This was strongly brought out in the construction of a duplicate sewer system. When the storm sewers are built in streets where the old sewer is in the center of the street, with the water main on one side and the gas main on the other, the city naturally locates its new sewers not where they will disturb the municipal water pipe, but on the side of the gas main, with the result that much of the gas main must be relaid. This makes constant trouble and expense for the gas company. It is also unfortunate that most of this sewer work is let in small jobs to irresponsible contractors, who have no interest in properly supporting or backfilling their trenches, so the neighboring gas pipes have to suffer. This trouble from storm sewer building is avoided in new streets now being piped by not laying the gas mains until the sewer and water mains have been laid. The gas main is then located between the two, a good location for the gas pipe, if the sewer and water trenches have been carefully backfilled. Unfortunately hundreds of miles of older streets were built and piped before the city had adopted the duplicate system of sewers. Accordingly there is trouble ahead for several years.

A long step towards avoiding these troubles would be made if it were possible to devise some wise system for assigning proper locations for all sub-surface structures in all new streets; but even if such a scheme were in force serious trouble would probably develop unless these structures were laid in their proper sequence, so those laid shall not be undermined by subsequent work.

Under present methods space is wasted by one party taking more than necessary and, by claiming unnecessary clearance for convenience, forcing other pipes into contracted areas. For instance, the parallel clearance demanded by different water departments ranges all the way from 6 to 12 ins. in one city to 5 ft. in another.

Some have the unfortunate habit of locating their pipes halfway between existing pipes, with the result that the next fellow has the choice of two contracted locations and, whichever he chooses, he will crowd both his neighbors; whereas if his predecessor had been wiser there would have been room for all.

If some experienced, fair-minded official or board could have the power to determine just where all pipes and conduits should be located, so that each structure could be given proper location, conditions would improve greatly. This official should be familiar with the laying and maintaining of such structures so as to intelligently locate them for the best interests of all concerned, and should be free from all bias or prejudice. Such a position would be no sinecure and would require tact and horse sense to adjust conflicting interests and do justice to all.

In Boston we have had a sample of the possibilities of this, one of the engineers of the transit commission being assigned the duty of supervising the relocating of all pipes and conduits over the new subways. This engineer had previous experience on telephone conduits and pneumatic tubes, but, best of all, the laying of gas mains, so his success was assured. The method now in force in Boston for obtaining a permit for a new main is as follows:

The engineer of distribution compiles a plan showing all existing structures in the street and indicates the desired location of the new pipe. A blueprint is sent with the application to the office of the commissioner of public works, and the location is approved successively by division engineers in charge of sewer, water, grades and catchbasins, and finally by the one in charge of highways, who indorses the application and the permit clerk issues the permit. This supervision is to protect municipal structures, regardless of its effect on the interests of other corporations.

Unless there is a thoroughly settled policy as to the locations to be given certain pipes, a change in the personnel of the departments may make radical changes. We have had executives in the street and park departments who located gas mains in sidewalk in preference to the paved or macadam street, while their successors changed the policy and kept all the pipes in the roadway. Unless some settled policy is in force it is impossible intelligently to locate outlets for branch streets when a new main is laid.

Every corporation or municipal department in a growing city is sure to outgrow the capacity of its original pipes and conduits, and the locating and laying of reinforcing pipes is a serious problem. But above all other things the problem of maintaining existing pipes in a safe condition is the most serious one for the gas company, and it is not always the last fellow in the street who has the worst of it. Everyone knows that an undermined cast iron pipe, is liable to settle and either leak or break, and this is especially dangerous when a manhole is built so as to inclose or support the pipe, while the adjacent lengths are depending on soft earth for support. The pipes may be so covered or crowded by new structures that it is practically impossible to caulk the joints, and large bills may be incurred for damage to the electric cables by driving bars and picks when making necessary repairs.

From my investigations of conditions in some of the larger cities I often find the solution of this problem is a local one, in that conditions vary.

As to the permanent right of the gas mains, it is conceded that a gas company has few that the city is bound to respect, and any gas lines which interfere with work of the city or state must be changed at the expense of the company. In many cities gas companies can collect from contractors on city work who damage their pipes, and in one city the gas company is recompensed by the city for moving its mains on account of subway construction. In a few cities the gas company is paid actual cost for moving its pipes, and occasionally a gas company bills the city for damages to gas pipes due to carelessness.

In some cities the locations are not assigned or approved, while in others such assignment or aproval is made after plans are submitted by the gas company. In many cities there is a well-established understanding as to the location of the sewers and water and gas mains, while the location of the electric conduits is more a matter of chance.

The relative locations vary in different cities. In one large city the water main is the center of the street, where most often we find the sewer. In another the center of the street is given to the gas main. This seems a most happy condition, unless the street is wide enough to require two lines, or the center of the street is occupied by car tracks. From this location services can be run to either side without blocking the street or requiring the pipe to be laid in two sections.

In another important city the mains are laid between curbstones and line of street when practicable. Most gas men select a location in the roadway near the curb line, and if there are car tracks there should be two mains. Philadelphia has as near a standard system of locations as any of the large cities, and locations for underground structures are assigned by the board of highway supervisors consisting of director of public works and the chiefs of the highway, city property, survey and water bureaus.

These standard locations are generally followed: Gas, 3½ ft. from curb; telephone, 5 ft. from curb; sewer, 7 or 8 ft. from curb; electric railroad conduit, close to either rail; water, in center of street.

This plan of standard locations often receives a setback other than due to existing structures installed before the plan was inaugurated, owing to the fact that the three city bureaus which install underground structures lay pipes or conduits without consulting the board of highway supervisors, and take any location they prefer. Thus any system will prove a failure if the city departments, responsible only to themselves, ignore its provisions. We could hardly conceive of a corporation being allowed to break up a system in that way.

The authorities of one very large city have recently attempted to establish a bureau of sub-surface structures, designed to locate and map all such so they will be able to assign proper locations for all sub-surface companies, insuring permanence and preventing interference with existing structures. This is the theory, but I am informed that to the present time about 1/7 of 1 per cent of the territory has been plotted, so we may confidently hope the grandchildren of the present generation may enjoy its benefits. A similar scheme was tried in Boston about 25 years ago, but after the original sectional plans were made there was no appropriation to keep them up to date, and they soon became of little value, so now our plans must be compiled by collecting data from all known to have pipes in the street.

A gas company with which I was connected was required to file plans annually at the city hall, showing locations of all gas mains laid; but I doubt if they were consulted by the departments for whose benefit they were made, as the draughtsman came to the gas office to compile the location plans for water pipes and sewers.

It may be practical for some modern cities to start such a system and derive practical benefit from the data on file; but for an old city, where gas pipes have been laid annually since 1824, its systems of water pipes and sewers being entitled to the respect of old age, it is not an easy thing without a large annual expenditure, and for this reason such schemes are apt to come to an untimely end.

Operation of the Philadelphia High-Pressure Fire Service System.

The management and operation of the Philadelphia high-pressure fire service system was transferred from the department of public service to the department of public works in the spring of 1912. The service was then placed in charge of the bureau of water in accordance with the policy of the department of public works of combining in one bureau all pumping stations. Some features of the organization developed for handling the system and of the results secured are here given from information taken from the latest annual report of the bureau of water.

The high-pressure system, with the extensions put in service during the year, covers the business portion of the city between Walnut and Race streets, from Broad street to the Delaware river, and the mill district of the city lying in general between Allegheny avenue, Girard avenue, Germantown avenue and Front street, with extensions embracing certain outlying sections of this mill district. It comprises 49 miles of pipe ranging in size from 8 ins. to 20 ins., and is provided with 877 hydrants. Two stations supply water to the system up to a maximum pressure at the station of 300 lbs.

Station No. 1, located at Front and Race streets, and drawing water from the Delaware river, contains seven 1,200-gal. pumps and two 350-gal. pumps, having a combined capacity of 9,100 gals. per minute. This station serves the business district of the city.

Station No. 2, at Seventh street and Lehigh avenue, drawing water from the Fairhill reservoir, contains ten 1,200-gal. pumps and one 350-gal. pump, with a combined capacity of 12,350 gals. per minute. This station serves the mill district of the city.

Both stations are equipped with three-cylinder four-cycle gas engines. While each station serves a distinct district, the two districts are tied together, so that in case of an emergency either district can be fed from either station by two separate routes. The manipulation of the valves necessary to make these changes requires but a few minutes.

During the year Station No. 1 was in actual service for periods ranging from 20 minutes to 48 hours for 23 fires. There were 81 additional calls in which the line pressure was built up and held, but in which the system was not needed. Station No. 2 was called into service 31 times, on four of which the service was used.

Station No. 1 consumed during the year 2,175,260 cu. ft. of gas for all purposes, including experimental runs. Station No. 2 consumed 357,600 cu. ft. of gas for all purposes, including experimental runs.

Upon the service being taken over by the department of public works the organization was divided into three parts, having distinct duties:

1. *Maintenance.* Under an assistant engineer acting as assistant superintendent.
2. *Operation.* Under a superintendent in direct charge of the operations of the stations.
3. *Construction and Engineering.* Under the first assistant engineer as engineer in charge. This force concerns itself with engineering and construction work.

Under the maintenance force a repair gang is organized for each distribution district. Each gang is in charge of a foreman, and is provided with a light motor truck for carrying the men to and from their work. The use of these motor trucks has improved the service, decreased the time necessary for the manipulation of stop valves in case of a leak, and in responding to fires.

Each line is put under pressure for one hour weekly and tested for breaks, unless the line has been in service under working pressure during the preceding seven days.

During the year thirteen broken flanges were found on the lines in the business district. The pipes in this district are cast-iron flange pipes laid about 1902. On the lines in the mill district there were found one 8-in., three 12-in. and two 16-in. split and broken pipes. This pipe is Universal cast-iron pipe laid during the last three years.

Investigation of these split pipes seems to show that the whole movement of the pipe, due either to the working pressure or the expansion in the mains after they were laid, accumulated in one joint, forcing the taper end forward until the pipe split under this additional pressure.

When the high-pressure system was taken over by the department of public works the loss of static head, due to leaks, etc., amounted to about 16 lbs. per square inch. By a systematic campaign of going over these lines, testing, retesting, replacing hydrants, valves, etc., this loss has been reduced to 5 lbs. per square inch.

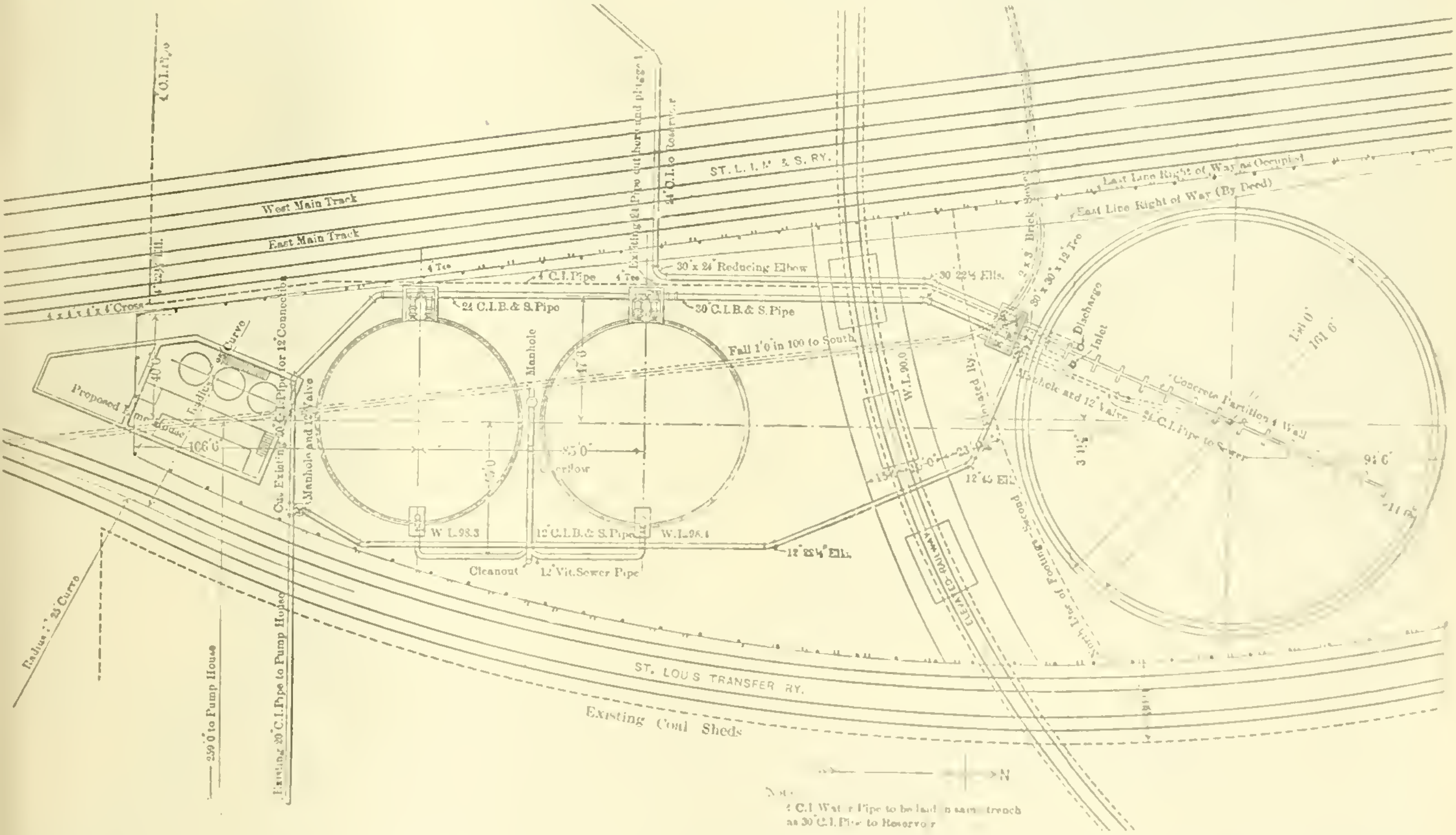


Fig. 1. General Plan of Old (Steel) and New Reinforced Concrete Settling Reservoirs at Anheuser-Busch Brewery, St. Louis, Mo.

A disputed point between the fire bureau and those in control of the high-pressure system as to the maximum limit to be placed upon the pressure in the mains in case of fire has been definitely settled. A conference, attended by city and fire department officials, and of insurance representatives decided that 250 lbs. was a safe limit to be placed on the pressure at the hydrants, except in case of extraordinary emergency. It was agreed that an increased pressure carried with it serious danger of blowing out the system at several points at once, thereby depriving the fire bureau of the entire use of the high pressure.

It was also agreed that this pressure was sufficient to supply all the water that could be controlled or would be required at any fire except a serious conflagration.

Short Course in Highway Engineering at the University of Illinois.—A short course in highway engineering is to be given by the University of Illinois, Jan. 19 to 31. In addition to lectures upon the various features of road and bridge construction, and the selection and operation of gasoline engines and tractors, demonstrations of road building machinery will be made by several manufacturers.

Design of Reinforced Concrete Settling Reservoir and Coagulation Plant for Anheuser-Busch Brewing Association at St. Louis.

While the city of St. Louis reserves the right to supply its citizens with water it has no power to prevent the owners of riparian rights from taking water from the river and conducting it to industries on the river front, provided that the private pipe lines do not enter or cross any public thoroughfare or other publicly owned lands. Thus there has developed at the Anheuser-Busch brewing establishment a water supply system with a capacity of more than 6,000,000 gals. per day, or sufficient to supply the average city of 80,000 inhabitants. The principal features of this water supply system are here described from information taken from the paper prepared by Edward Flad, entitled: Reinforced Concrete Reservoir and Coagulation Plant at St. Louis, Mo., for presentation before the American Society of Civil Engineers on Feb. 1, 1911, and published on pp. 2133 *et seq.* of the Proceedings for December, 1913.

The waterworks of the Anheuser-Busch Brewery are on the river front, south of Dorcas street. The water is taken from the Mississippi through two 20-in. cast iron intake pipes, and is siphoned into one of two intake wells, from which it is pumped into settling tanks. It flows by gravity through the settling tanks, thence through the filters, and is then pumped into the distribution system. A general plan of the settling tanks is shown in Fig. 1.

The low-service pumps are in a brick pit, 30 ft. in diameter and 40 ft. deep. There are three centrifugal pumps having a combined capacity of 13,000,000 gals. per day, and one triplex, direct acting pump having a capacity of 2,000,000 gals. per day.

There are two steel settling tanks, 75 ft. in diameter and 28 ft. high, one circular concrete reservoir approximately 150 ft. in diameter and 30 ft. deep, and one rectangular covered reservoir having a capacity of about 1,000,000 gals.

The filter plant comprises six Jewell filters, circular in plan, each 16 ft. in diameter, and three Keisert filters, recently completed, which are rectangular in plan, each being approximately 34 by 15 ft.

Chemical Treatment.—The water is settled by adding sulphate of aluminum (alum) and lime. A special three-story reinforced concrete building is provided for storing and preparing the chemicals.

The hopper for storing the lime is 36 by 9 by 14 ft. high, and has a capacity of 90 tons. It is placed in a pit so that it can be filled directly by shoveling from the cars. An electric elevator conveys to the third floor the hand-cars containing the lime or alum.

The alum is dissolved in a concrete tank, and is fed to the water by gravity. This alum tank has three rectangular divisions, each 10 by 7 by 5 ft. deep. Each division is charged with from 500 to 2,000 lbs. of alum which is dissolved in water. It requires from two to five hours to dissolve one charge. The lime is slacked in iron tanks on the third floor. These tanks are rectangular, 12 by 12 ft. by 7 ft. deep with sloping bottoms. A false perforated bottom is provided at a depth of 30 ins., on which the lime is placed and partly submerged in 1 ft. of water. After slaking, which requires about one hour, the attendant stirs the mixture, which passes readily through the perforated bottom and into the lime tanks below. This milk of lime is stored in three vertical cylindrical iron tanks, 12 ft. in diameter and 18 ft. high, with conical bottoms. One charge of lime consists of from 2,000 to 6,000 lbs., and the tank holds about 10,000 gals. of water, giving a 2½ to 7 per cent solution. In these tanks the milk of lime is kept agitated by compressed air admitted at the bottom through a small perforated pipe.

The flow of both the alum and lime solutions is regulated by standardized orifices operating under fixed heads. The milk of lime is pumped into the supply pipe or settling tanks by a centrifugal pump. As a general rule the lime is added to the water as it enters the first settling tank, and the alum as the water enters the second settling tank.

Consumption.—The maximum consumption of water during the summer is about 6,000,000 gals. per day, for which, at times, 1,600 lbs. of alum and 5,000 lbs. of lime are required, or about 2 grains of alum and 7 grains of lime per gallon of water.

The water is used for boiler purposes, for condensing and cooling in connection with the ice machine, as well as for washing barrels, bottles, etc. It is not used in brewing, partly because of the natural prejudice against water taken from the river below the outlet of the large city sewers.

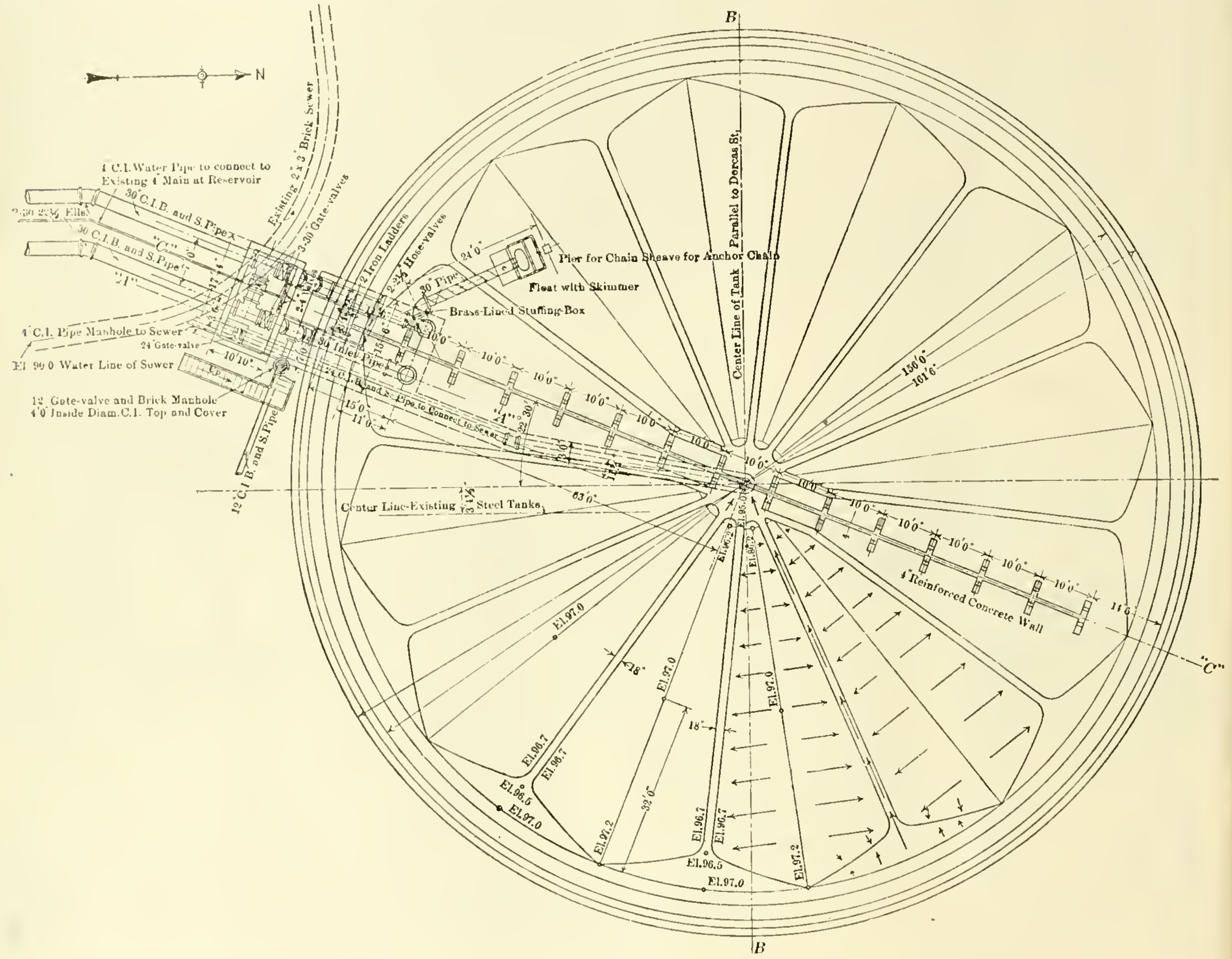

N
4 C.I. Water Pipe to connect to Existing 4 Main at Reservoir
2-30 22½ Ells
30 C.I.B. and S. Pipe
30 C.I.B. and S. Pipe
"C"
"21"
Existing 2 x 3 Brick Sewer
3-30 Gate-valves
2 Iron Ladders
2-2½ Hose-valves
30 Pipe
24'0"
Pier for Chain Sheave for Anchor Chain
Float with Skimmer
Brass-Lined Stuffing-Box
4 C.I. Pipe Manhole to Sewer
24 Gate-valve
El. 90.0 Water Line of Sewer
10'10"
Up
12 Gate-valve and Brick Manhole
4'0" Inside Diam. C.I. Top and Cover
12 C.I.B. and S. Pipe
30 Inlet Pipe
24 C.I.B. and S. Pipe to Connect to Sewer
15'0"
11'0"
"A"
63'0"
Center Line-Existing Steel Tanks
3 1/2
Center Line of Tank Parallel to Dorcas St.
156'0"
161'6"
10'0"
4" Reinforced Concrete Wall
11'0"
"C"
B
B
El. 97.0
18"
El. 97.0
El. 97.0
18"
32'0"
El. 96.7
El. 96.7
El. 96.5
El. 97.0
El. 97.2
El. 96.7
El. 96.7
El. 96.5
El. 97.0
El. 97.2

Fig. 2. Foundation Plan of New Reinforced Concrete Settling Reservoir—Anheuser-Busch Brewery, St. Louis.

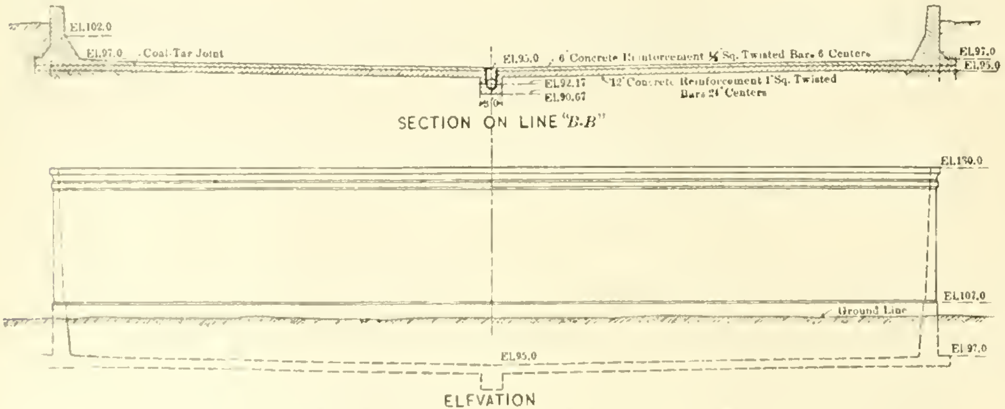


Fig. 3. Sectional and Side Elevations of New Reservoir—Anheuser-Busch Brewery.

Reinforced Concrete Reservoir.—Reverting now to the reinforced concrete reservoir, the large settling tank shown in Fig. 1, recently completed: After due consideration of the various possibilities as to the shape and location of the reservoir, a circular shape was decided on, and the diameter was made as large as the available space permitted. The elevation of the top was fixed by that of the water in the old settling tanks, which operate in series with the new reservoir; and the bottom was placed sufficiently low to pass below the fill of cinders and rubbish, and rest on the river silt.

Estimates were made of the comparative cost of a steel tank and a reinforced concrete reservoir. Exclusive of the foundations, pipes, gate-house, and accessories common to both designs, the steel tank was estimated to cost $32,000, and the reinforced concrete reservoir, $30,800. The reinforced concrete reservoir was supposed to have some advantage, being a more permanent form of construction and not requiring painting, and perhaps a desire to follow the latest fashion had a minor influence; at all events it was decided to use reinforced concrete.

The reservoir has vertical sides and is 153 ft. 6 ins. in diameter at the top and 35 ft. deep at the center. The side-wall extends 25 ft. 6 in. above the ground. The capacity is approximately 4,250,000 gals. There is a central partition, consisting of a 4-in. reinforced concrete wall with buttresses, which starts at one side of the reservoir and passes diametrically across to within 14 ft. of the other side. The object of this partition is to make the entering water circulate around the reservoir before reaching the outlet. The diameter of the intake and outlet pipes is 30 ins., and that of the waste pipe 24 ins. The outlet pipe has a float and a hinged joint, so that water is always taken from near the surface. The valves conrolling the flow are in a gate-chamber outside the reservoir.

Foundation.—The foundation is a 12-in. layer of concrete resting directly on the river silt and reinforced, in two directions at right angles with 1 in. square bars, 2 ft. from center to center, making ½ per cent reinforcement each way.

On top of this foundation rests the bottom of the reservoir, which is 6 ins. thick and reinforced in a manner similar to the foundation, except that the bars are ½ in. square and 6 ins. from center to center.

The top of the foundation was coated with a thin layer of coal-tar, the object being to provide for expansion and contraction of the bottom independent of the foundation.

Side-Walls.—The thickness of the concrete side-walls, shown in Fig. 4, is 7 ft. 6 ins. at the base, 2 ft. 5 ins. at the ground line, and 12 ins. at the top. The pressure of the water is carried by hoop tension, the side-walls being reinforced circumferentially with corrugated round bars, under the assumption that the concrete carries no tension. There are three lines of 1¼-in. round bars at the bottom and two lines of ¾-in. round bars at the top, the sections and spacing being varied from bottom to top to correspond with the pressure. It was intended to allow a stress of 15,000 lbs. per sq. in. on the bars. Owing to an error in the calculation, which, happily, was on the safe side, an excess of steel was used, making the stress per square inch somewhat less than that originally contemplated.

The thickness of the concrete wall was fixed at each point so that the concrete would not be stressed more than 290 lbs. per sq. in., under the assumption that no vertical cracks would develop under this tension. Assuming the modulus of elasticity of steel at 30,000,000 and concrete at 3,000,000, the actual maximum stress, if no vertical cracks develop, will be 2,900 lbs. per square inch of steel and 29 lbs. per square inch of concrete.

The following formulas were used in determining the dimensions and spacing of the steel bars and the thickness of the concrete:

A = Area of bars, in square inches per foot of height of wall;

D = Vertical distance, in inches, between two layers of bars at the point selected;

T = Thickness of concrete wall, in inches, at the point selected;

p = Water pressure, in pounds per square foot, at the point selected;

r = Radius of reservoir, in feet;

s = Stress in steel, in pounds per square inch, allowed under the assumption that the concrete carries no tension;

c = Stress in concrete, in pounds per square inch, allowed.

a = Area, in square inches, of steel in each layer.

The area of the bars required per foot of height of wall is

$$A = \frac{p\,r}{s}$$

The vertical distance between the layers of bars is

$$D = \frac{12\,a}{A}$$

The thickness of the concrete wall at any point is

$$T = \frac{p\,r - 9\,A\,c}{12\,c}.$$

The reinforcing bars were held in position by angle-iron frames, the sides of the angles being punched accurately with small holes for the insertion of the wires which tied the bars to the frame.

Splices.—The circumferential bars were in lengths of from 50 to 55 ft., and were spliced by lapping 40 diameters and attaching two Crosby clips at each lap. The strength of this joint was tested by making up sample joints and pulling the bars in a testing machine. Two tests were made on each diameter of bar used, one with bars and clips without any mortar, and the other with the same joint embedded in cement mortar of the same mixture as that used for the walls of the reservoir.

The tests showed that the presence of the mortar did not add materially to the strength of the joint, as it failed either by slippage or by breaking off the clip. The first sign of slippage occurred at a stress of from 18,000 to 35,000 lbs., and the ultimate strength of the joint varied from 25,000 to 75,000 lbs.

Table 1 gives the results of the tests.

At the junction of the side-wall with the bottom, knee-bars are provided to reinforce this connection. These bars are 1 in. square and 12 ins. from center to center.

Mixture.—The concrete in the walls and bottom is a mixture of 1 part cement, 1½ parts sand, and 3 parts limestone screenings run through a ¾-in. mesh screen, with the addition of 10½ lbs. of Shamrock water-proofing to each barrel of cement. The composition of this water-proofing is approximately as follows:

Silica	60.0 pct.
Alumina	15.0 pct.
Lime	6.5 pct.
Oxide of iron	1.5 pct.
Combined water	10.0 pct.
Gelatinous material	7.0 pct.

This material cost 73½ cts. per barrel of cement.

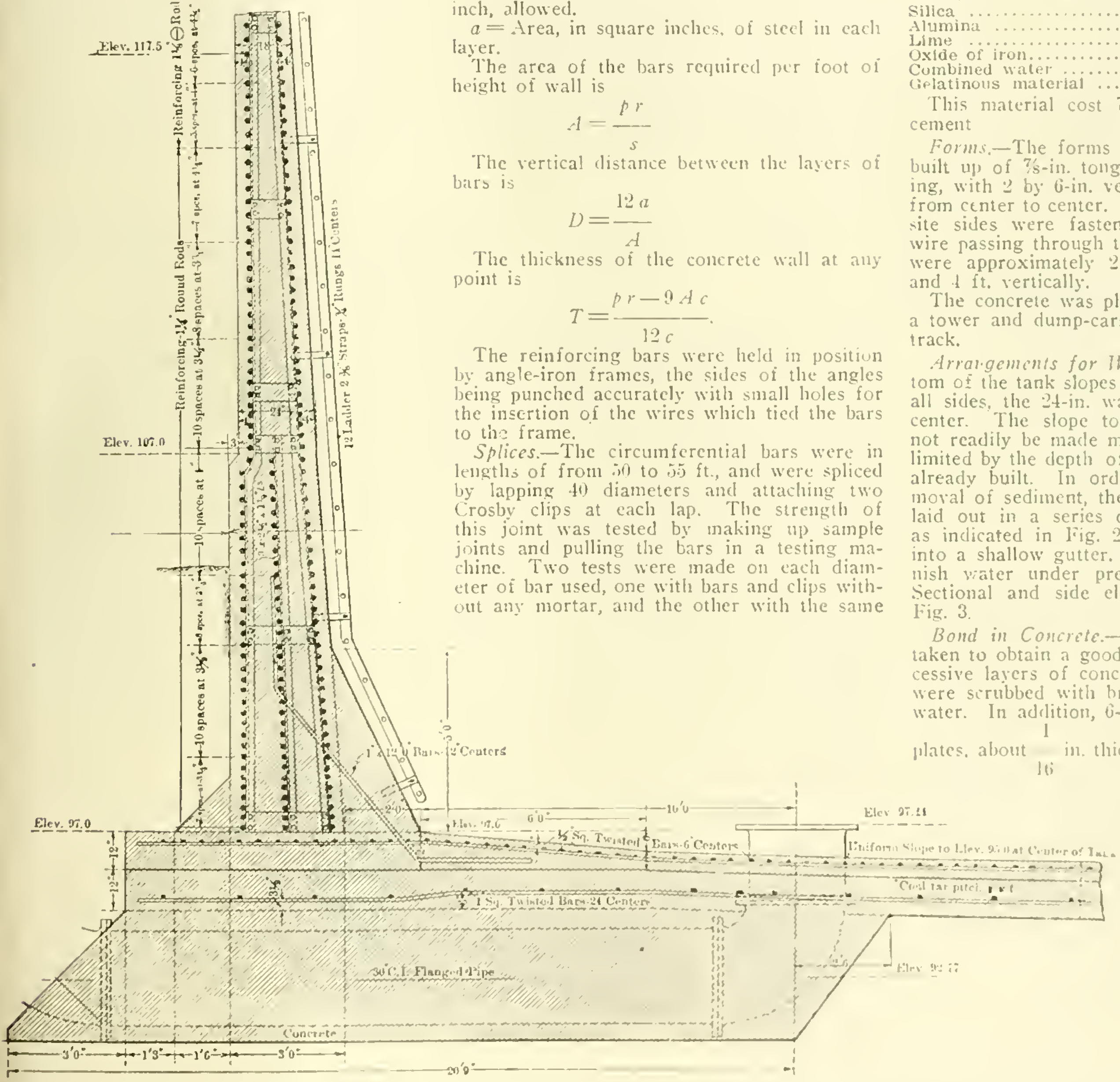
Forms.—The forms for the side-walls were built up of ⅞-in. tongued and grooved flooring, with 2 by 6-in. vertical studs about 2 ft. from center to center. The studs on the opposite sides were fastened together with iron wire passing through the forms. These wires were approximately 2 ft. apart horizontally and 4 ft. vertically.

The concrete was placed in the forms with a tower and dump-cars running on a circular track.

Arrangements for Washing Out.—The bottom of the tank slopes toward the center from all sides, the 24-in. waste outlet being at the center. The slope toward the center could not readily be made more than 2 in 75, being limited by the depth of the sewer which was already built. In order to facilitate the removal of sediment, therefore, the bottom was laid out in a series of star-shaped mounds, as indicated in Fig. 2, each mound draining into a shallow gutter. Hose connections furnish water under pressure of cleaning out. Sectional and side elevations are shown in Fig. 3.

Bond in Concrete.—Special precaution was taken to obtain a good bond between the successive layers of concrete. The old surfaces were scrubbed with brushes and a stream of water. In addition, 6-in. strips of corrugated plates, about $\frac{1}{16}$ in. thick, were placed vertically in the joints. In spite of these precautions, when the reservoir was filled, small leaks developed along most of the joints, and efflorescence was quite extensive. These leaks appear to be closing up gradually, and it is probable that in the course of time they will disappear entirely. The intention, however, is to empty the reservoir and treat the joints with some water-proofing compound.

After completion the outside of the wall was



Elev. 117.5
Reinforcing 1¼ ⊖ Rods
Reinforcing 1¼ Round Rods
10 spaces at 3½ 8 spaces at 3½ 7 spcs. at 4½
10 spaces at
10 spaces at 3½ 8 spcs. at 2½
Elev. 107.0
12 Ladder 2 × ⅜ Straps ⅞ Rungs 11 Centers
1 × 12 0 Bars 12 Centers
5'0"
2'0"
Elev. 97.0
Elev. 97.0
6'0"
10'0"
Elev. 97.41
½ Sq. Twisted Bars 6 Centers
Uniform Slope to Elev. 95.0 at Center of Tank
Coal tar pitch joint
1 Sq. Twisted Bars 24 Centers
30" C.I. Flanged Pipe
Elev. 92.77
Concrete
12"
12"
3'0"
1'3"
1'6"
3'0"
20'0"

Fig. 4. Vertical Section of Wall and Footing of New Reinforced Concrete Settling Reservoir of Anheuser-Busch Brewing Association, St. Louis, Mo.

TABLE 1.—TESTS OF BARS

Kind of bar	Began slipping	Broke at	Remarks.
1¼-in. round bar, plain	......	73,600 lbs	1-in. slip at 18,000 lbs.
1¼-in. round bar, with mortar	......	10,500 lbs.	Mortar broke at 29,000 lbs.
1⅛-in. round bar, plain	27,100 lbs	......	
1⅛-in. round bar, with mortar	32,200 lbs	72,200 lbs	
1-in. round bar, plain	35,000 lbs.	......	
1-in. round bar, with mortar	[illegible]	[illegible]	
¾-in. round bar, plain	27,300 lbs	[illegible]	
¾-in. round bar, with mortar	26,100 lbs	26,400 lbs.	

rubbed with carborundum blocks and brushed with cement mortar.

The reservoir was designed by the writer's company, and was built under contract by the Fruin-Colnon Contracting Co., of St. Louis. The cost, including pipes and accessories, was approximately $52,000. The unit prices and total cost of the various classes of work were as follows:

Excavation, 7,801 cu. yds. at 80 cts.	$ 6,240.80
Plain concrete, class A, 58.2 cu. yds., at $4	232.80
Plain concrete, class B, 281.3 cu. yds., at $5	1,406.50
Reinforced concrete, 12-in. base, 759 cu. yds., at $5	3,795.00
Reinforced concrete, 6-in. bottom, 473.1 cu. yds., at $8	3,784.80
Reinforced concrete side walls, 1,178.3 cu. yds., at $11	12,961.30
Reinforced concrete partition, 120 cu. yds., at $20	2,400.00
Reinforcing bars, 582,668 lbs., at 1¾ cts.	10,196.69
Clips for reinforcing bars	650.00
Ladders and angle-iron supports	852.16
Cast iron pipes and valves	7,283.00
Gate house	1,200.00
Miscellaneous	844.59
Total cost	$51,847.64

The Break in the Water Supply Conduit of Montreal—Method of Making Repairs.

(Staff Article.)

A serious "blow-out" of a portion of the reinforced concrete conduit which conveys the municipal water supply of Montreal occurred on Dec. 25, 1913, and until Jan. 2, 1914, when the conduit was again successfully placed in service, the city was plunged into the confusion and distress incident to a partial water famine. An official report on the cause of the break and the general condition of the conduit is being prepared, at the instance of the proper city officials, by J. A. Jamieson and P. W. St. George. The gist of that report will be presented to our readers when it becomes available for editorial use. Pending the receipt of the report mentioned we have prepared the following statement relative to the failure of the conduit and to the methods employed in repairing it. The present article is based on information derived from various sources.

Section of Conduit.—Figure 1, herewith, shows a cross sectional view of the concrete conduit which failed. Figure 2 is a diagrammatic cross section showing the relative position of the old and new canals and the conduit.

The smaller canal shown in Fig. 2 was formerly used to conduct the water supply of the city from the St. Lawrence River. About five years ago the reinforced concrete conduit was built and since then the work of enlarging the first canal has been prosecuted at intervals. The new canal is to furnish water for power and industrial purposes. The primary cause of abandoning the old canal was that the water it carried was exposed to serious contamination.

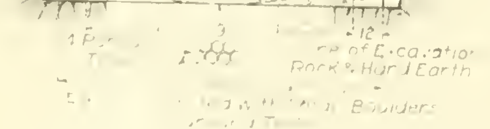


Fig. 1. A Typical Cross Section of Reinforced Concrete Conduit, Montreal Water Supply System, Which Failed Dec. 25, 1913.

The old canal had a width at the water line of 80 ft. and a depth of 8 ft. The old canal was widened to 140 ft. and its depth increased to 14 ft. under a contract awarded in 1910. Work on the canal widening was begun in 1877, long before the enclosed concrete conduit was thought of, but a change in plan soon led to the cessation of excavation operations and they were not resumed until after the concrete conduit was in use.

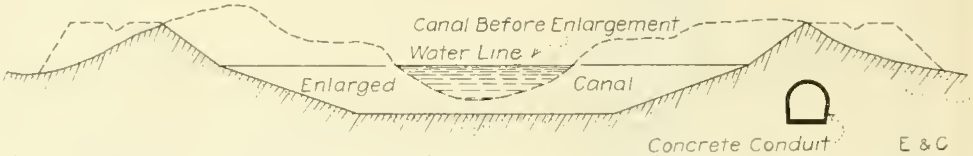


Fig. 2. Montreal Water Conduit—Sectional Diagram Showing Relative Positions of Old Canal and Its First Enlargement and the Water Supply Conduit.

(The second enlargement of the canal, which was under way at the point where the blow-out occurred, is 4 ft. deeper and 34 ft. wider than largest canal above shown.)

As shown in Fig. 1 the conduit is of the horseshoe type, the radius of the invert being 13 ft. The greatest inside height of the section is 7 ft. 3½ ins. and the greatest width is 9 ft. The inside width of the bottom segment is 7 ft. 2½ ins. The thickness of the concrete at the top and bottom is 8 ins. and at the level of the greatest width of section it is 12 ins. The concrete is 1:2:5 mixture reinforced with No. 7 and No. 12, Birmingham gage, Clinton wire mesh, the larger strands being 4 ins. and the latter 12 ins. apart. The soil traversed by the conduit ranges from limestone to clay. The stability of the latter soil is dependent upon its moisture content. Where the surrounding soil was unstable the reinforcement extends entirely around the conduit section. Where the foundation was stable the reinforcement was carried 1 ft. within the outer ends of the bottom segment of the section. The size of stone specified for this construction was up to 1½ ins. in greatest dimension for the portion below the greatest horizontal width of the section and up to ¾ ins. in the balance of the section. It was further specified that the wire cloth should not at any point be closer than 3 ins. to the inner surface of the conduit. The capacity of the conduit is 50,000,000 imp. gals. per day. The length is 27,300 ft. and the grade 1 in 5,000. When carrying the normal flow the hydraulic grade line is only from 5 to 10 ft. above the top of the conduit.

The larger canal, shown in Fig. 2, was completed under a contract awarded in 1910, and almost as soon as this work was finished a further widening and deepening of the canal, to 171 ft. and 18 ft., respectively, was decided upon. This final enlargement of the canal section has been carried on by the Cook Construction Co. The new canal bottom is 5 ft. lower than the invert of the concrete conduit at the point where the accident occurred, and at that point the canal side slope is only 17 ft. from the conduit.

About the middle of December the contractor had about excavated the canal section to its full size at the point where the break in the conduit subsequently occurred. The excavation, which was in clay at this point, was made by means of a drag line machine which moved parallel to the canal. This excavating machine was a short distance from the conduit, the conduit being between the scraper and the canal. The scraper was on about 5 ft. of spoil from the canal excavation. This made the depth of filling over the conduit about 10 ft. At the time mentioned it was noticed that the conduit was leaking, so excavation operations were suspended. The leaking portion of the aqueduct was exposed and the cracks through which the leakage occurred were calked. Stone and sand for filling were also deposited back of a line of sheet piling set between conduit and canal. This repair work was completed two days before the blow-out occurred but, as developments indicate, it was of no avail.

A strip of the conduit concrete 60 ft. long and 2 ft. wide, on the canal side, gave way on the evening of Christmas day. The conduit was, of course, soon emptied and Montreal was without an adequate supply of water until the evening of January 2. The conduit head gates were closed as soon as the break occurred.

METHOD OF MAKING EMERGENCY REPAIRS.

A local iron works was given a rush order to get out seven 18 ft. lengths of riveted steel pipe, 7 ft. in diameter and ¼ in. thick. While the pipe was being made the entire section of the conduit was torn out for the full length of the break. The bucket of the drag line excavator previously mentioned was used as a battering ram to break out the concrete. It took almost two days to clear away the portion mentioned.

The steel pipe was placed in the conduit in such a manner that it extended across the break and 32 ft. farther at each end. Adjacent sections of the pipe were bolted together by means of angle iron flanges, which were riveted to the inside of the pipe. The placing of these flanges inside instead of outside the pipe caused embarrassment later on, as will be described.

The entire length of lap between the concrete and the steel pipe was filled to form a seal. At the ends of the steel pipe bulkheads were laid up, between the pipe and conduit, of brick in bitumen. The remaining portion of the seal was made by concrete deposited around the pipe through holes drilled for that purpose through the top of the conduit. Use was also made of cement in bags, presumably for the outer ends of the lap, and of grout applied by the cement gun. An effort was made to hasten the setting of the concrete by introducing live steam into the conduit and by banking coke fires against the exposed portion of the pipe.

As soon as the seal was completed as described the head gates were opened and it was at once apparent that the repairs made were inadequate. Leaks occurred through the seal and also through the bolted flange joints which were provided with rubber gaskets. Since the bolts were inside the pipe it was necessary to empty the conduit in order to tighten them. Men entered the conduit through a manhole in the exposed portion of the pipe line and tightened up the bolts. The seal between pipe and conduit was also improved, and the new concrete, which was carefully placed, was given 24 hrs. in which to set before the gates were again opened. This time the repair was a success. Since the head gates were opened on the evening of January 2 the normal supply of water has been delivered through the conduit to the city.

While the municipal conduit was out of service the city was partially supplied with water through connections between the city mains and the mains of the distribution system of the Montreal Water and Power Co. About 30 per cent of the normal supply was furnished in this manner. Some water was distributed by means of portable tanks on carts.

Indiana Engineering Society.—The annual meeting of the Indiana Engineering Society will be held Jan. 22, 23 and 24 at the Hotel Severin, Indianapolis, Ind. In connection with the convention there will also be an exhibit of engineering material by various manufacturers and so forth. The annual banquet will be held Jan. 22, at 8 p. m. and an invitation is extended to all of those interested in engineering work in Indiana to attend.

Roads and Streets

Methods and Cost of Constructing an Experimental Clay-Macadam Road at Pomeroy, Wash.

Contributed by J. E. Tupper, City Engineer, Pomeroy, Washington.

In common with good roads enthusiasts hroughout the country the writer has been tudying the theory and practice of construcion of hard road surfaces as fully and careully as circumstances would permit for the ast ten years. Previous to that time my work had been along other lines and my attention had not been called to the subject xcept in a casual way. In pursuing this tudy every text book and technical magazine rticle bearing on the subject that I could et hold of was given careful consideration, without any bias in favor of one method nore than another. Especial attention was iven to what may be termed non-technical rticles appearing in newspapers and agricultural journals, as I have frequently found hat the comments of non-technical men often ontain points that are well worth heeding y the technical man.

The conclusion to which I was forced to ome was that there is no unanimity of theory r practice among road engineers, and further hat such rules as seem to be generally accepted are purely empirical. There are a few undamental principles that are so simple as o be axiomatic, applying equally to all roads egardless of the surfacing materials, but beond this there seems to be practically no ata that can be said to be scientifically established.

No sooner does it seem to be thoroughly stablished that concrete is the one and only eliable foundation for a pavement than Mr. Hammatt appears in Western Engineering or September with an article in which he dvances the heretical argument that conrete has not filled the bill, and even goes o far as to say that "As a substitute for oncrete I believe that water bound macadam annot be improved upon." Shades of Byrne! What next? Listen: "Water bound macadam as but two faults as an ideal pavement for nything but congested city traffic. One of hese is its tendency to disintegrate under notor traffic, and the other is its permeability o water." This is but a sample of what we re meeting with continually, and in the face f such conditions we can scarcely denounce s heretical any new proposition, however adical it may seem to be, nor need we fear o attack any of the old rules of procedure imply because they have gained general aceptance if there appear to us to be a fundamental defects in their position.

When in 1911 I was called upon as county ngineer to prepare plans and specifications or the construction of a mile of water bound macadam road I realized that to a very great xtent the future of road improvement in ur county was at stake. Should the road ail in any respect to meet the requirements would be a long time before another experiment could be made. The sentiment of he public was very largely either violently ntagonistic or indifferently neutral. The ounty board was unanimous in opposition to he expenditure of money for any [illegible] purose, and it was only by reason of the fact hat under the state law a fund had been created which could be used for [illegible] purose that they were finally [illegible] to conent to the construction of the road. The oad selected for the experiment was one unning east from the city of P[illegible] along he axis of a deep valley, a[illegible] hroughout its entire length [illegible] wind—a condition admittedly h[illegible] macadam construction. The travel over the road s heavy at all times, averaging [illegible] vehicles per diem, and, there being no parallel oad that would serve the purpose, the travel ould not be diverted and must be taken care of during construction. So far one could hardly conceive of harder conditions under which to work.

As a partial offset to these conditions I was able to get specifications approved which gave me a very free hand in working out the problems that might arise, the foundation for the road was, in the main, exceptionally good, and we secured a contractor who heartily co-operated with me in my endeavor to secure the best practical results.

A quarry of very excellent trap rock was opened which was well located for handling the product by gravity, and exceptionally accessible to the road. The rock is very hard and tough, has a theoretically good fracture, but is wofully lacking in binding qualities, whatever that may mean. At least I have been informed by experienced road engineers that the failure to get it to form a compact mass without the addition of some extraneous material was due to that fact.

CROSS-SECTION.

As this is one of the most important roads in the county, and carries as much traffic as any two other roads, it was thought best to give it ample width, and yet there did not seem to be any good reason for metalling a wider surface than was needed for convenient passing of vehicles. The state law provides for a minimum width of 12 ft., which appears to us to be an anomaly; being too wide for a single track and not wide enough for teams to pass on. Sixteen feet was chosen as being the most suitable width for the macadam surface. As there was much travel over the road with bare-footed horses and considerable objection in general to the stone surfaced road, we felt obligated to make the earth shoulders wide enough so that the travel could follow them in case the macadam surface was objectionable. As a general proposition I am convinced that this is a good plan in any case. Making the earth shoulders 8 ft. wide instead of 4 ft., as is usual, gave us a width of roadbed of 32 ft. In order to obtain this and have room for broad ditches with flat slopes we made our road 40 ft. wide in cuts. With this construction ample room is given for the travel to keep off of the macadam surface, and the efficiency and desirability of the road is considerably enhanced without any disadvantage other than the increased cost of grading.

The travel will not follow the earth surface during muddy weather, nor to any great extent during the dry season on account of the dust, but during very much of the year the conditions are favorable for travel on the earth surface and the light traffic will follow along the side of the road to the considerable saving of the metal surface. In [illegible] 16 ft. as the width of the metal surface we were governed solely by the needs of the traffic.

A very large part of the traffic over the road consists of heavy loaded teams, and provision must be made for the meeting of the teams without forcing the heavy loads off the macadam onto the earth shoulders. Sixteen feet is the minimum width that will do this, and yet is not too wide to allow of reasonable width to the earth shoulders without making the total width of the road too great.

In my investigation of the [illegible] for water [illegible] [illegible] sive luxuries [illegible] during the [illegible] of rolling the earth will recede under the pressure of the crushed rock and the result will be a deformed cross section similar to that shown by Fig. 2, instead of the theoretical form shown by Fig. 1. This sharp triangular edge with its point on the surface is bound to crush off as soon as the earth surface begins to wear away or to soften in wet weather. Some have attempted to correct this defect by hand placing a row of Telford blocks along each edge, thus forming a sort of rough curb. This device helps for a time in that it prevents the formation of the sharp edge and consequent early ravelling; but the ultimate result must be much the same, and perhaps even worse. As the earth wears away or becomes soft the wheels in passing from the metal to the earth will have an abrupt drop, the blow from which will hasten the formation of a deep rut, destroy the earth support for the blocks and finally displace them, producing a worse condition than in the first case. Others have tried placing 2x6-in. plank along the edges to hold the stones in place. The only advantage this has over the curbstone method is in the cost, while the final result must of necessity be very similar. The more I studied the subject the more was the conclusion forced upon me that the whole vertical shoulder method was fundamentally unsound. The earth shoulders could never be depended upon to hold the edges of the crushed rock in a vertical position. Eventually the edges would break down and flatten out until they had assumed such a form as they could retain of their own inherent stability. In doing this the adjacent portions of the macadam surface must suffer more or less. Why not give the rock such a form during construction as it could be depended upon to retain without the aid of external support. The idea appealed to me forcefully, and though I could find no support in any of the authorities I decided to try the method and watch the results. Later on, during the course of the construction, a member of the contracting firm handed me a little book by H. P. Gillette entitled "Economics of Road Construction" in which similar arguments were used and a similar method of constructing recommended. While starting from the same premises, using the same arguments, and, in the main, arriving at the same conclusions, we have developed cross sections that are in some respects radically different. In Fig. [illegible] is shown Mr. Gillette's cross section while mine is shown by Fig. 1.

Mr. Gillette discards all side protection for the macadam, leaving the necessarily raw edges bare, and makes the earth shoulders [illegible] continuation of the subgrade. Seemingly, he does this on the grounds of providing better drainage, and it appears to me that he bases his argument on a fallacy. He apparently assumes that the subgrade is more important than the macadam [illegible], and that some means should be provided by which the water that penetrates the macadam surface may drain away before it has time to penetrate the subgrade. [illegible] road men [illegible] the idea [illegible] essential [illegible] macadam, [illegible] has [illegible]

[illegible]

[illegible] width

of the roadbed. The central 17 ft. on which the macadam is to be placed is then trimmed with the road grader by starting at the center line and throwing the dirt outward and giving the surface a crown of approximately 2 ins. The additional crown is secured by making the metal thicker in the center than at the sides. After placing and spreading each course of stone, and before rolling, a couple of rounds are made with the grader, bringing dirt from the sides up against the stone until the earth surface is somewhat higher than the stone. This excess will usually disappear under the action of the roller, but if it still remains it is easily removed after the rolling is completed by the final trimming with the road grader that should be given in any case. The result will be a continuously smooth surface, with no evident line of demarkation between the stone and the earth surfaces, and absolute protection of the edges of the macadam surface against shearing by the wheels of heavy loaded vehicles, a protection that will not be destroyed by the wearing down or softening of the earth shoulders until that wear equals the thickness of the metal. Before that time the material worn from the shoulders will have been replaced if any reasonable care is taken of the road. In actual practice this theory has been absolutely sustained.

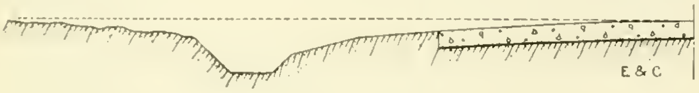


Fig. 1. Earth Shoulders as Generally Planned for Macadam Roads.

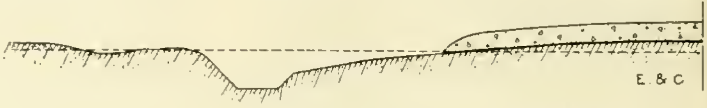


Fig. 3. Cross-Section of Macadam Roads Recommended by H. P. Gillette.

The transition from the metal to the earth surfaces has been imperceptible at all times, there has been absolutely no shearing or ravelling of the edges, and no ruts have formed over any part of the surface of the road. From a cost standpoint the saving is considerable. The work was done as a part of the grading contract and the actual cost to the county is not easily ascertained. The actual cost to the contractors was $9 per mile. Taking as a basis the costs given in Gillette's "Hand Book of Cost Data" and reducing to the basis of the wages paid here the cost of preparing the subgrade where the trenches are called for cannot be less than $350 per mile, and would probably cost $500 per mile. It may be, and probably was true, that the saving accrued to the contractor in this case. Even then the public has benefited and eventually will be able to secure lower prices if this method is generally adopted.

CLAY BINDER FOR MACADAM.

We had but fairly got started with the construction of the road when a new and wholly unforeseen problem confronted me.

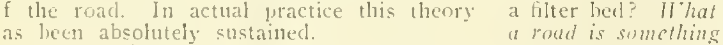


Fig. 2. Earth Shoulders as Generally Constructed for Macadam Roads.

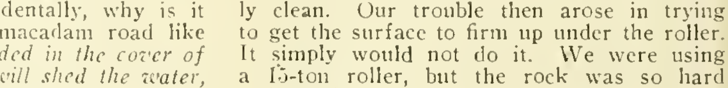


Fig. 4. Cross-Section of Macadam Roads Recommended and Constructed by J. E. Tupper.

We were taking our rock from a ledge of very much shattered basalt, and the crevices were filled with clay that had washed in from the overlying soil. Because of the shattered condition of the rock the output of screenings was not very great, even including the clay content, which amounted to fully 50 per cent. What to do with this "mud" was the problem. Our "Missourians" did not believe in putting mud on a road that was being built to get rid of the mud. Work had begun late in the fall and frequent rains kept the screening pile so wet that it was useless to try to get rid of the clay by the use of an extra finer screen and washing would be difficult and costly. Closing down the crusher so as to get a larger output of screenings would only partially remedy the trouble and would largely increase the cost of the crushed rock. At last I decided to allow the use of the screenings as they came from the crusher on the first course of macadam, hoping that as the quarry was opened up the clay content would be reduced. About 1,000 ft. of the first course was laid in this way and finished and 600 ft. of the second course was spread but not yet rolled when the weather got so bad that we had to suspend operations until spring. During the winter, while work was suspended, it was utterly impossible to keep the travel off of the unfinished road. Of course they made ruts in the partially finished surface, and equally of course they carried quantities of mud on this surface and beat it into the portions traveled over. Whatever the effect of this travel on the ultimate condition of the roadbed might be, the matter was one that was beyond our control and we must abide by the results. During this period, while we were waiting for the weather to settle so that we could resume operations, I studied as never before the question of why clay need be objectionable in road construction, why it was necessary to have the crushed rock clean and free from any earthy matter? If these are true, then there must be reasons for them, and these reasons I failed to find. What is mean in "binding qualities" in stone, and what is needed and can reasonably be expected in a binder? Incidentally, why is it necessary to construct a macadam road like a filter bed? *What is needed in the cover of a road is something that will shed the water, not strain it.* It is self-evident that in some manner the stones in the macadam surface must be made to retain their position, and the more certainly, readily and cheaply it can be done the more serviceable and economical will be the construction of the road. It seems probable, though by no means certain, that with some kinds of stone, such as the limestones, shales, and stone containing a considerable amount of iron ore, that there may be a limited amount of chemical action akin to that occurring in the set of Portland cement as an aid in binding the mass together. With the inert rocks like granite, quartzite, and basalt this cannot be conceded, and the action must be purely mechanical. There seems to be an idea prevalent in the minds of engineers that with the right kind of crushed rock a sort of knitting and interlocking action takes place under the pressure of the roller similar to the felting action of wool or fur. As a matter of fact no such action takes place, as may readily be seen if the nature and form of the material is taken into consideration. What does take place is the formation of a more or less perfect matrix for each individual particle under the crushing and grinding action of the roller, and the greater the susceptibility of the rock used to the formation of dust and well graduated fine particles out of which the matrix can be formed, the more perfect the so-called binding qualities of the stone. The limestones and shales excell in this quality while the traps are notably deficient. It may be set down as an axiom that the greater the binding qualities of any stone the less the wearing qualities. Now, why should any great weight be placed on the binding qualities of the stone in selecting the material for a road? If the formation of a matrix is what is wanted we have in clay a more perfect material than can be found in the dust and screenings of even the most perfectly ideal rock. While clay is naturally very cohesive and has excellent bearing powers, it does not stand abrasion or percussion well, especially when either very wet or very dry, but when thoroughly puddled it forms a firm bed for particles of sand, gravel or crushed rock, holding them in place while they take the brunt of the abrasive and percussive action of the feet of teams and the wheels of vehicles. That this is true is amply borne out by the success of the sand-clay roads that have been built in various parts of the country. Taken by itself, neither sand nor clay makes a good road surface. Sand will not remain firm and compact, and clay grinds up into dust in dry weather and into mud in wet weather. Combined the good qualities of each appear, resulting in a very good and cheaply constructed road. The clay carries the load, being protected from abrasion by the sand and gravel, which in its turn takes the wear, being held in place by the clay bed.

As the result of my studies I decided to continue the use of the clay in the binder, at least for a portion of the road, in spite of the volume of adverse criticism.

As was anticipated, the clay content of the screenings diminished as we got deeper in the quarry, so that by the time the first 1,000 ft. was completed the screenings were practically clean. Our trouble then arose in trying to get the surface to firm up under the roller. It simply would not do it. We were using a 15-ton roller, but the rock was so hard and tough that there was no evidence of any crushing or the production of dust under its action, and it simply would not set up. The roller was run continuously for three days on 800 yds. of clean stone in the attempt to get a firm surface, and it was no nearer complete at the end of that time than it was at the end of six hours' run. As at that rate the contractors were loosing money without our getting anything for their work, I directed them to haul clay from an adjacent bank, and spread it over the road, sprinkle it thoroughly, and see what the result would be. This was done and in a few hours we had what appeared to be a perfect surface. The test of service showed, however, that in this section of the road the clay did not penetrate deep enough to thoroughly bed the surfacing material with the result that it began to ravel soon and had to be given further treatments. From then on we treated the road to clay before attempting to give it a final rolling. After rolling the screened rock to as firm a surface as could be secured without a binder we spread a coating of clay over the surface, sprinkled it thoroughly to wash it down into the interstices of the rock, followed with a coating of rock screenings and sprinkled and rolled it until a satisfactory surface was secured. We found that in this way the screenings would work down into the voids of the rock, the wet clay would ooze up under the pressure of the roller, and the surplus clay would be washed away by the sprinkler. It was at about this stage of the work that I got hold of H. P. Gillette's book on the "Economics of Road Construction" and read with interest his contention that trap rock could not be made sufficiently firm for a road surface without the addition of clay, road dust, or loam to the binder. As the result of our experiments with binding materials on this road becomes more evident I feel justified in stating positively that not only is it necessary to use some such addition to the screenings as an aid to their binding qualities, but I am prepared to go even farther and to assert that the addition of clay in proper amounts to macadam work is essential to the best results. When I say clay I mean a good, tough, tenacious quality of clay suitable for brick making. Road dust or loam will aid in the setting up of the surface during construction, but the effect is not lasting. When the traffic comes on the road such binders break down, crumble up and blow away. Another result of the use of clay, and in my estimation by no means a matter of minor importance, is the fact that the addition of clay renders the surface practically impervious to water. Ask any hydraulic engineer and he will tell you that a clay puddle is about the most impervious substance known to the profession.

METHODS OF CONSTRUCTION.

While we are about it, let us break a few more precedents. I set no grade stakes or cross lines to distribute the stone by. What is the use? If the subgrade is tested up as it should be all that is needed is to have the stone brought on in loads of uniform size, which is easy to do where dump wagons are used, and dump the loads at uniform intervals. Your distribution will then be as uniform as it would be with all the stakes and chalk line gages you can place. I have the loads dumped directly on the roadbed, placing the wagon squarely across the road when dumping. The terrible bug-a-boo of hard places in the road where the loads are dumped has no terrors for me. They do not materialize. I had stakes set at 8-ft. intervals for the first course and 16 ft. for the second course and required 2 cu. yd. loads to be dumped at the stakes. Of course, with automatic spreading wagons a somewhat different course would have to be pursued, but in either case I should stake out the dumping intervals and insist on their being observed. The spreading was done first with what is locally known as a Morman scraper, followed with a heavy harrow. While the harrowing was being done the man who was in charge of the spreading attended to the alignment of the edges, corrected any faults that might develop during the harrowing by filling low places and removing material from high spots, and tested up his crown as needed with a straight edge. A good man is needed in this place. Much of the justifiable criticism of our road can be traced directly to ignorance or carelessness on the part of the man we had in this place. If careful work is done here there will be little need of corrections following the roller.

Under the first contract let we constructed 5,700 ft. of two-course macadam, the first course consisting of stone retained on a 1½-in. screen and not exceeding 4 ins. in any dimension; the second course consisted of stones passing the 1½-in. screen and retained on the ⅜-in. screen. The first course was distributed at the rate of ¼ cu. yd. per linear foot and the second course at the rate of ⅛ cu. yd. per foot. Allowing for a compression of 29 per cent this produced a final thickness of 6 ins. Before the first contract was completed two additional contracts were let for extensions of the road aggregating 3,500 ft. In these contracts the two sizes of rock were run together, only the screenings being taken out. Otherwise the procedure was the same as in the first: It is hard to tell which makes the best road. The metal for the second contracts was considerably denser and did not compress as much as the first lot. The result is a rather thicker surface coat and one that appears to wear rather better.

COST DATA.

As this work was all done under contract and the force available in the engineer's office was insufficient to keep continuous tab on the costs it is not possible for me to give a full analysis of the costs of the different parts of the work. The following cost data were taken during one entire week when the force was thoroughly organized, the men all familiar with their work, the weather ideal and everything working smoothly. It represents the maximum output of the force, and is considerably in excess of the average. The output of the quarry averaged 75 cu. yds. a day exclusive of the screenings, or 87 cu. yds. including screenings, bunker measure. The face of the quarry at the time was about 21 ft. high and 12-ft. holes were drilled in the face close to the bottom. All drilling was done by hand, one man holding and one striking. The blasts were fired with electric detonators in batteries of three or four holes, each hole carrying 15 lbs. of 60 per cent dynamite and breaking down 10 cu. yds. per foot of hole or 8 cu. yds. per pound of powder. This very exceptional showing was largely due to the shattered condition of the ledge. The mucking was easy, there being very little sledging required. A 24-in. gage track carrying ½ cu. yd. steel dump cars was installed, running from the crusher platform to the face of the quarry, with a switch and two branches. The cars were loaded and operated by hand.

The daily quarrying and crushing costs during the week observed are given in Table I.

TABLE I. DAILY COSTS OF QUARRYING AND CRUSHING SHATTERED TRAP ROCK, POMEROY, WASHINGTON.

	Per day.	Per cu. yd. bunk. meas.
Engine rental	$ 5.00	$0.0667
Engineer	3.50	0.0466
Gasoline and oil	4.00	0.0533
Quarry foreman	3.50	0.0467
2 drillers	5.00	0.0666
8 muckers	20.00	0.2666
1 feeder	2.50	0.0333
Supervision	2.00	0.0267
Powder	1.95	0.0260
Detonators	0.12	0.0016
Total	$47.57	$0.6341

The crushing was done with a No. 2 Aurora portable crusher driven by a 25-HP. gasoline engine. The crushed rock was elevated to a two section revolving screen, dropped into hopper bottomed bunkers, and spouted directly into the dump wagons. Three teams hauled the rock from the bunkers to the road in 2 cu. yd. dump wagons, one team hauled screenings and clay in a 1½ cu. yd. wagon box, the teamster and one helper loading and spreading it as needed. One team was kept constantly at work on the sprinkling wagon, and one team spread the rock and harrowed it, helping at odd times with the sprinkling. Two extra men were kept on the road assisting in distributing the materials and correcting defects that appeared under the action of the harrower or roller.

The roller, a 15-ton Austin gasoline, operated by one man, was kept constantly at work. Only for the fact that the harrowing reduced by fully 30 per cent the amount of rolling needed, the roller would have had to work two shifts much of the time. With the force as above stated they completed 200 lin. ft. of road per eight-hour day at the unit costs given in Table II.

This is equivalent to a cost of 30.1 cts. per sq. yd. of surface. It must be borne in mind, however, that the cost given in Table II is the actual minimum cost possible with the force as organized, and includes no item for overhead charges or profits.

SERVICE AND MAINTENANCE.

This road was completed at the beginning of harvest in 1912 and has now passed through two seasons of the heavy traffic incident to the delivery of the grain crop. The portion of the 1913 crop that passed over this road during the months of August, September and October amounted to 15,000 tons, an average of 600 tons per day. Though no accurate traffic census has been taken on this road, it is known that during the busy season the travel often reaches 1,500 vehicles per 24 hours. Much of this traffic consists of heavy loads and it is not unusual to find instances where the load amounts to 1,000 lbs. per inch in width of tire, which is double the pressure given by our road roller.

At first, either through ignorance or indifference, no provision was made for the maintenance of the road and the turning of this heavy traffic on the green surface could only result in serious damage unless care was taken to prevent it. The result of the heavy traffic soon began to manifest itself and the surface began to shown signs of abrasion, especially in places where an insufficient amount, or a poor quality, of clay had been used. The authorities soon found that they must do something or the road would be a dead loss. We had no road roller, nor were we prepared to crush rock. The only thing possible to us under the circumstances was done.

We had more clay hauled and distributed where the road showed signs of failure, and then put a sprinkling wagon on the road and gave it a thorough soaking. This stopped the damage, and by sprinkling the road twice a week during the dry season we were able to prevent any further damage. This year the sprinkling was begun as soon as the dry season set in, and, aside from the damage above mentioned, which was due to faulty construction and to neglect while the road was still green, there has been no further damage sustained worth mentioning. As to the effect of motor traffic on the road it does not seem to do as much harm as do the caulks of the sharp-shod trotting teams. By watching the condition of the surface and keeping it sufficiently moist so that no considerable depth of dust will form there will be nothing for the tires to pick up and no damage can be done. This sprinkling, that has cost us $60 per mile per year, is all that has been done so far in the way of maintenance. At a fair estimate $100 per mile will repair the damage done when the road was new. Of course, there has been some wear on all parts of the road, but a careful examination made a short time ago shows that this does not exceed ½ in. except in the places above mentioned. If this rate can be maintained the road will last ten years without resurfacing.

TABLE II. UNIT COSTS OF CONSTRUCTING CLAY-STONE MACADAM ROAD, POMEROY, WASHINGTON.

	Per day.	Per lin. ft.
Quarrying and crushing	$ 47.50	$0.2375
Hauling stone	13.50	0.0675
Hauling screenings and clay	4.50	0.0225
Team spreading stone	4.50	0.0225
Team sprinkling	4.50	0.0225
3 extra men	7.50	0.0375
Roller operator	3.50	0.0175
Gasoline and oil	5.00	0.0250
Rental of 3 dump wagons	1.50	0.0075
Rental of 2 sprinklers	2.00	0.0100
Rental of road roller	10.00	0.0500
Supervision on the road	3.00	0.0150
Total	$107.00	$0.5350

ECONOMIES OBTAINED AND COMPARISON WITH CONCRETE ROADS.

The question naturally arises, does it pay to build a road that will cost $100 per mile a year for maintenance, and that will require resurfacing at the end of ten years at cost equal to one-half the original cost of the road? Why not build a concrete road in the first place and save the cost of maintenance at least? The question is one that will not down and it must be answered.

Both the county and the city are doing considerable concrete work of various kinds and the cost of concrete road construction has been pretty thoroughly investigated. The least we can hope to lay a concrete pavement 16 ft. wide for, is $1.50 per square yard. We can place the best macadam for $0.50 per square yard at the most. Roughly speaking the cost of macadam is $5,000 per mile and the cost of concrete is $15,000. This gives an excess cost for concrete of $10,000 upon which interest must be computed. Placing this at 4 per cent, which is lower than we can reasonably expect to get the money, we have an annual interest charge of $400. Even the most optimistic advocates of concrete road construction admit an annual maintenance charge of $25 per mile, and most of them concede $50 per mile. Then the surface is not indestructible, and though concrete has not been used long enough for this purpose to enable us to form more than a shrewd guess, it is fair to assume that taking one road with another they will require renewal at the end of 20 years. For this purpose at least $500 must be set aside, making in all an annual charge of $925 per mile on the concrete construction as against $350 for the macadam.

CONCLUSIONS.

The proof of the pudding is in the eating, and the real and only true test of the value of a road is the estimation in which it is held by that portion of the public which uses it. As I stated at the beginning of this article, the feeling of the public was at first largely antagonistic, but before the first half mile had been completed the feeling had changed to so great an extent that the farmers who would use the road contributed $2,625 for an extension of the road 2,700 ft. farther east, and the city of Pomeroy appropriated $700 for an extension into the city. Fifteen of the subscribers paid $100 each and the verdict of these men is unanimous that they were amply compensated by the use of the road the first year. No macadam work has been done this year, nor is any contemplated for next year that is to be paid for out of the permanent highway fund.; but this is not because of any dissatisfaction with that type of road or doubt of its ability to stand the wear. We are simply waiting until the accumulation in the fund is sufficient to warrant the letting of larger contracts, and in the meantime we are getting our road in shape out of the other funds to receive the metal surface.

The growth of public sentiment in favor of improved roads is fairly mirrored in the attitude toward taxation. At the time this road was started the levy in this county for road purposes was four mills on the dollar of valuation, which was about the average of the preceding ten years. This was nearly all needed for the ordinary repairs and very little was available for permanent work. The next year the levy was raised to six mills, which proved to be woefully inadequate to the demands. This year the call for road improvement was so great that the board raised the levy to the statutory limit of 14 mills, an increase of 350 per cent in two years. It may not be fair to attribute the whole of this enthusiasm to the construction of this one piece of road, but as it was the first piece of good road constructed in the county it had much to do with it. A movement is on foot to surface 1½ miles of road in another part of the county with macadam 9 ft. in width, the expense to be borne largely by subscription. More than that, the success of the macadam on this piece of work has been such that the feeling is very strong in favor of using it for city pavements. It seems quite probable that 12,000 sq. yds. of macadam pavement will be laid in the city this coming season.

Among the lessons that we have learned from this experiment are that a water and clay-bound macadam road kept well sprinkled will stand up under any traffic; that it is in most cases cheaper and better to keep a sprinkling wagon on the road during the dry season than to attempt to keep down the dust and hold the metal in place with a bituminous binder; that the better the quality of clay used in the binder the more durable the road; that where a good quality of clay is used there is no danger of failure of the road through saturation of the sub-grade; and that a well built macadam road is superior to either asphalt or concrete as a road surface.

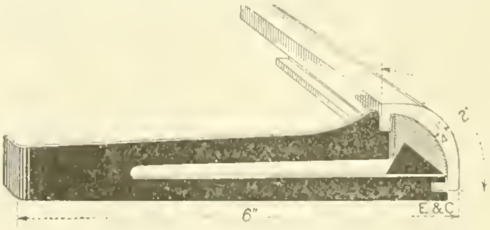
In this article I have not attempted to cover all of the experiments that we made in the course of the construction nor enumerate the problems we had to solve. Many of these were of minor importance and had to do mainly with the efficiency of the working force. Others affecting the quality of the road showed negative results. In all of our experiments cost was made subordinate to quality, and a method was not rejected on account of cost until it was clearly established that it would be true economy to do so.

It is not argued that the methods we used were such as would produce the most perfect surface. Some small depressions developed soon after traffic was turned on the road, the surface was somewhat wavy, and there was a tendency for the large stones to show up on the surface and make it rougher than it should be. Had we been constructing a city pavement we would have used greater refinement of detail, but the object we were striving for was the securing of a good, serviceable road, and we placed greater stress on stability than on ideal smoothness of surface.

An Improved Clip Anchor for Metal Guards for Concrete Curbs and Columns.

(Staff Article.)

In our issue of May 21, 1913, we published an illustrated description of a metal device for protecting the corners of concrete curbs and columns as made by the Clip Bar Manufacturing Co., 2540 Olive St., Philadelphia, Pa. The same manufacturers have recently placed an improved type of clip anchor on the market for use with their type of bar. The new style of clip anchor is shown in the accompanying view, which also shows the shape and dimensions of the bar which have not been changed. The new clip anchors have a double function: (1) To position and support the guard bar in the form while the concrete is being placed, and (2) to anchor the curb guard securely in the concrete structure when finished. The slip anchors are attached at 9-in. intervals by springing them into the bar. After all of the necessary clips are attached to the bar, it is placed in the form and the guard bar and clips are held in position by wedging the anchors between the two sides of the form which form the back and face of the curb. The guard bar can be bent to any radius of curb desired at street and alley intersections. Among the advantages claimed by the makers for this type of bar are the following: (1) Light strong section; (2) simplicity of installation; (3) efficient service; (4) great wearing surface; (5) ornate appearance; (6) not affected by frost; (7) not affected by expansion and contraction on account of uniform thickness.



An Improved Clip Anchor Metal Guard for Concrete Curbs and Columns.

National Conference on Concrete Road Building.—The program of the sessions of the National Conference on Concrete Road Building to be held at the Auditorium Hotel, Chicago, Feb. 12-14, 1914, will be as follows:

February 12: "The National Conference on Concrete Road Building," by Dr. W. F. M. Goss; "Financing Permanent Roads," by S. E. Bradt; "Can a Rural Community Afford Permanent Roads?" by Oliver Dunlap; and "The Concrete Road System of Wayne County, Michigan," by Edward N. Hines.

February 13: "Development of Concrete Roads in the United States," by H. C. Shirley; "Contraction and Expansion of Concrete Roads," by R. J. Wig, Ch., N. H. Tunnicliff and W. A. McIntyre; "Joints for Concrete Roads" by Dr. W. K. Hatt, Ch., G. W. Cooley and R. J. Wig; "Methods and Cost of Repairing and Maintaining Concrete Roads," by E. N. Hines, Ch., J. S. McCullough and F. P. Wilson; "Preparation and Treatment of Subgrade for Concrete Roads," by Prof. I. O. Baker, Ch., A. R. Hirst and A. N. Johnson; "Reinforcement of Concrete Roads" by T. H. McDonald, Prof. H. E. Riggs and R. L. Humphrey; "Experiments with Concrete for Roads Conducted by the United States Office of Public Roads," by Logan Waller Page; "Aggregates for Concrete Roads," by S. E. Thompson, Ch., Prof. A. N. Talbot and W. M. Kinney; "Shoulders for Concrete Roads," by Maj. W. W. Crosby, Ch., C. A. Kingsley and J. H. Mullen; "Bituminous Surfaces for Concrete Roads," by E. J. Mehren, Ch., H. G. Shirley and J. R. Marker; "Finishing and Curing Concrete Road Surfaces," by Dr. Turnsaure, Ch., H. J. Kuelling and E. D. Boyer; "Economic Methods of Handling and Hauling Materials for Concrete Roads," by H. P. Gillette, Ch., D. D. Price and P. H. Wilson.

February 14: "Concrete Road Construction by the Ohio State Highway Department," by James R. Marker; "Mixing and Placing Materials for Concrete Roads," by P. D. Sargent, Ch., Prof. A. H. Blanchard and C. W. Boynton; "Cost of Constructing Concrete Roads," by A. N. Johnson, Ch., Dr. J. H. Pratt and Albert Reichmann; "Thickness, Crown and Grades for Concrete Roads," by Prof. L. C. Smith, Ch., E. R. Whitmore and Prof. T. R. Agg; "Proportion and Consistency of Materials for Concrete Roads," by C. U. Bowley, Ch., C. C. Widener and G. A. Dingman; "Form of Specifications for Concrete Roads," by Dr. A. Marston, Ch., Prof. A. N. Talbot and G. W. Cooley.

Meeting of Committee to Consider the Improvement and Securing of Uniform Road Laws.

A definite effort is to be made to revise the road laws of the various states with a view to codifying and simplifying them and making them more uniform. A committee of the American Highway Association has been appointed to take charge of the matter and this committee has held its first meeting with C. T. Terry, of New York, who is chairman of the committee on uniform legislation of the American Bar Association. The members of the American Highway Association's committee are F. E. Wadhams, of Albany, N. Y.; A. B. Fletcher, State Highway Engineer of California; A. N. Johnson, State Highway Engineer of Illinois; P. T. Colgrove, President of the Michigan State Good Roads Association; and J. E. Pennypacker, Secretary of the American Highway Association. Mr. Wadhams is chairman.

Short Course of Civil Engineering Lectures for Highway, County and City Engineers and Officials.—During the week of Jan. 12-17, 1914, a short course of lectures and laboratory demonstrations on highway construction, highway bridges, highway maintenance, drainage, plain and reinforced concrete construction, testing of construction materials, and a special conference on road laws was given before a civil enginering conference of county surveyors, county commissioners, road superintendents, city engineers and other public officials at Purdue University, LaFayette, Indiana. The purpose of the course was to familiarize those attending the lectures with the results of tests, and to point out the right and wrong methods of doing public work. The lectures were given by members of the civil engineering faculty and prominent experts on subjects of interest to public officials.

Holing Through of Mount Royal Tunnel. The headings of the Mount Royal tunnel of the Canadian Northern Railway terminal project, at Montreal, holed through at 7 a. m., Dec. 10, within an hour of the time at which it had been estimated on Nov. 1, that they would meet. Over 17,000 ft. of heading have been completed in about fifteen months' time, or at the rate of progress of 1,100 ft. per month, for all headings. Since June 1, at which time a world's record for hard rock tunneling of 810 ft. of 8x12 heading, in thirty-one working days, was set, the two remaining headings have advanced towards each other at the rate of 960 ft. per month. Both headings have been in exceedingly hard rock, namely, cyanite, essexyte and volcanic breccia, for nearly all of this last period.

Engineering and Contracting

Devoted to the Economics of Civil Engineering Design
and to Methods and Cost of Construction

Volume XLI. CHICAGO, ILL., JANUARY 28, 1914. Number 4.

The Building Contractor—His Problems and Methods.

Contracting companies have not often made public the detailed methods used by them in organizing for and in carrying out their various contracts. The result is that there is considerable mystery surrounding the work of contractors, and owners and engineers have in some cases looked upon such men with suspicion. However, conditions are changing, and it is being realized that an efficient organization and open dealings with owners and engineers are important assets of a company, and that the publication of construction methods used on particular jobs constitutes legitimate and profitable advertising. Further, the publication of this information should tend to cause engineers to adopt types of construction which can be erected economically. Some firms have failed because they did not appreciate the necessity of good business methods—others have not possessed sufficient flexibility in their organization to meet the changing character of work. Some have engaged in more work than their capital and equipment justified, and thus were not able to provide for emergencies. The willingness of some contractors to take work on a speculative basis has not given the best results, either to the contractor or to the owner, as both are entitled to a fair return on their investment.

Many companies are now glad to take work on a cost-plus-percentage-basis, which simply means that the speculative element is omitted and that only a fair return is expected. With this form of contract the contractor rents his equipment and organization and becomes the owner's broker in the purchase of materials; while the owner furnishes the capital required to provide for depreciation of the contractor's plant, the cost of labor and a fair profit. Contracting companies working on this basis must rely mainly upon their ability to give good service and to do reliable work in order to insure future contracts.

In our issue of Jan. 14, 1914, we gave an abstract of a part of a paper by Mr. Leonard C. Wason, president of the Aberthaw Construction Co. of Boston. We are giving a rather full abstract of the remainder of this paper in this issue, and have been glad to afford it considerable space, believing as we do that conditions will be improved if engineers and owners better understand the contractor's problems and difficulties. Mr. Wason gives emphasis to what he calls the "human element" and states that it is highly desirable to thoroughly understand the temperament of the owner, architect and engineer if harmonious work is to be done. It is pointed out that contractors soon learn what treatment to expect from certain architects and owners and govern their bids accordingly. It is evident that the attitude of the owner and architect has much to do with the progress and cost of the work. The importance and the difficulty of securing good inspectors is pointed out. It is not sufficient to get an honest inspector, but it is necessary to get one who is entirely familiar with the particular kind of work which he is to inspect.

The company with which Mr. Wason is connected favors the cost-plus-fixed-sum for profit form of contract, as such a contract makes the interests of owner, architect and engineer identical. To emphasize the fact that the contractor for building work is largely a broker it is stated that in office building work the contractor may not handle more than 10 per cent of the contract price as direct labor, the remainder being for materials bought and for sub-contracts let. In reinforced concrete buildings the percentage of labor is about 35 per cent of the total cost.

The best results can be secured only by perfecting a smoothly working organization and confining the contracts to what this organization can handle. There is great risk in taking on too much work and in employing superintendents and foremen for a single job. If work is done in widely separated cities it can not be efficiently handled by the main office, as incorrect methods can not be detected and corrected soon enough to be effective. The system of carefully analyzing the design in the main office and then planning and routing the work has many advantages. The making of detailed drawings for the forms by men skilled in this branch of the work—instead of allowing the carpenters to lay out the forms—has the advantage that the lumber can be ordered in such lengths and sizes that cutting is largely avoided. Further, better results can be obtained by this method, as carpenters capable of doing the highest class of work can not often be obtained for concrete work. It is important that special study be given to determine the rate of progress necessary to complete the building on time. The crowding of work when not absolutely necessary will require greatly increased expenditures for equipment and labor. Most contractors appreciate the desirability of an efficient cost-keeping system. However, all do not realize that a system which merely enables the contractor to determine the unit costs of the completed work has little value. A system to be of greatest value must be one by which unit costs can be determined as the work progresses. Such a system enables those in charge to determine what part of the work is costing too much, thus enabling them to correct defects in time to prevent large wastes. In the main this paper deals with those common sense features which we are too apt to consider of little importance, and in addition mentions some new methods which by test have been found to be efficient.

The Engineer's Part in the Regulation of Public Utilities.

In the regulation of public utilities, the engineer is most often represented not by membership on the regulating board or commission but by employment by that body. Lawyers, accountants, business men and politicians take precedence of engineers when appointive power chooses the members of public service commissions. The appointment of an engineer is indeed so rare that it is considered to merit comment by the journals of his profession. Invariably, however, an engineer is appointed by the commission to direct its chief work, and many engineers are employed in his service. The conclusion follows that the engineer is considered a good servant but a poor master. Naturally engineers regard this as a radically wrong opinion.

Recently Dr Alexander C. Humphreys, President Stevens Institute of Technology, invited discussion of the subject "The Engineer's Part in the Regulation of Public Utilities." This discussion was participated in by Dr. Humphreys, Mr George Gibbs, consulting engineer; Mr. Newcomb Carlton, vice-president Western Union Telegraph Co.; Mr. John W. Lieb, vice-president New York Edison Co., and Mr James E. Sague, member Public Service Commission of New York (Second district).

In opening the discussion Dr. Humphreys called attention to the condition outlined in the paragraph above and contended that the engineering profession should be fully represented in every public service commission. He stated, however, that whether the commissioner is engineer, lawyer, accountant or business man he should have had experience in the exercise of administrative functions, and he pointed out that engineers were in part responsible for not deliberately and advisedly equipping themselves for administrative responsibilities. If membership of a public service commission must be confined to a single profession he considered that five or seven broadly trained engineers, with competent legal assistants on the staff, would give far better results than has on the average been obtained thus far by commissions composed as they are.

Referring to Dr. Humphrey's remarks, Mr Carlton pointed out that the engineer possessing sound technical knowledge coupled with broad administrative experience is in demand everywhere, and is thus placed beyond the call of public service commissions as they are now constituted. He considered the personnel of such commissions at present as far from satisfactory and that these commissions would be made far more equitable, judicial and competent by the appointment of trained experienced engineers as members.

The value of having as members of public service commissions technically educated men was considered by Mr. Lieb so obvious as to need no special proof. To him it seemed hardly creditable to the authority and influence of the engineering profession that its members played so unimportant a part in the personnel and organization of public service commissions. Most of the problems of such commissions rested upon the determination of such matters as physical valuation, functional depreciation, normal wear and tear, obsolescence, suppression, adequacy of equipment, betterments, etc., and this determination and valuation was an engineering task.

Following Mr. Lieb, Mr. Sague joined with him and previous speakers in expressing surprise that "engineers should have taken so little interest heretofore in state regulation which affects the business and success of practically all engineers. It is the right and duty of engineers to take part in the solution of these great problems. They can be settled properly only after the truth has been ascertained, and a fundamental duty of engineering is to ascertain the truth and to work in accordance with it."

The discussion by Mr. Gibbs, himself an engineer with experience on both sides of the railway regulation questions, was a distinct variation of thought from that expressed by preceding speakers. He considered that an engineer in the position of business manager or commissioner on a public utility board, on account of his multifarious duties, soon gets out of touch with the details of professional progress, and unless he is willing to delegate the investigation of engineering matters to subordinates who are specialists, his decisions must rest upon general knowledge which is not always a true guide. The matter of proper commission organization is fundamentally one of selecting faithful men of sound judgment and organizing ability, previous specialized training being a matter of less importance. The internal organization should include a permanent engineering staff headed by a chief engineer of eminence in his profession and the retention of outside engineering consultants who are specialists in the various lines of engineering advice needed and who are called in as occasion requires. This organization, of course, presupposes that the engineering staff will be called upon for analysis of the technical problems arising before the commission and that the latter will accept the advice of its staff upon such matters. The engineer has an important status in connection with the commission work, and, as a matter of fact, this is so well recognized that staff engineers are frequently more highly paid men than the commissioners themselves. The added honor of holding the title of "commissioner" is, of course, attractive, but is not essential to the dignity of the engineering profession, nor will the field opened up by the employment of a few engineer commissioners add materially to the opportunities of men of that profession in securing positions of high honor and remuneration in the community.

The thoughts which stand out upon reading the remarks which have been summarized are: (1) The general admission of the importance of the service in public utilities regulation of engineering knowledge and experience; (2) the general feeling that engineers have been apathetic and short sighted in not advancing themselves in responsible participation in public service regulation; (3) the query whether public service commissionership proper holds out any considerable enticement in the way of high popular honor or increased remuneration greatly attractive to engineers. Each of these subjects is one upon which a separate discussion is warranted and in succeeding issues we shall give such discussions. For the present we shall suggest only that engineers consider seriously the engineer's part in the regulation of public utilities as it has been outlined here.

Further Comment on the Increased Demand for Water Following the Introduction of Filtration.

In recent editorials we have called attention to the desirability of making ample provision in designing the capacity of filter plants for a rapid increase in consumption of water, following the introduction of filtration due to an increase in the number of people taking the city supply and to a freer use of the water. We have mentioned these facts in connection with two large cities in which the increase in water consumption following the placing in operation of filters has been so great that within a few months thereafter the construction of additional filters has been planned. Still further evidence of the increased popularity of a public water supply after filters have been constructed comes to us from a north central city which had a population of 70,000 in 1910. The name of the city is withheld since to give it would add nothing to the force of the illustration it affords of the point under consideration, and might direct criticism upon the designer who labored under we know not what limitations, financial and otherwise.

The filtration plant consists of twelve rapid sand filters of a combined normal rating of 12,000,000 gals. daily, three coagulating basins with a total capacity of 2,500,000 gals., equivalent to five hours as a maximum period of sedimentation, a clear well having an available filtered water capacity of 1,750,000 gals., six chemical solution tanks, arranged for feeding either iron and lime or alum as a coagulant and bleach for sterilization, and the usual wash water and sump pumps, air blower, etc.—in a word a thoroughly modern and efficient type of plant. The only criticism which attaches to the plant is its insufficient capacity. The plant is already overtaxed, at times, although it has been in operation less than two full years.

The plant was placed in service in the spring of 1912. It is illuminating to note the number of new services installed per year before and after the plant was placed in operation. During the years 1909, 1910 and 1911, 563, 651 and 716 new services were installed, respectively. In 1912, the first year of the filter's operation, 1,563 new services were installed and in 1913 the number of new services placed was 1,043. During the two years last named the total number of services in the city, which, of course, has long had a public water supply, increased exactly one-third.

The city in question is very inadequately metered. At the end of 1910 37 meters were in service and this number had been increased only to 53 meters at the end of 1913. There are no meters in service smaller than 1 in. and ordinary house services are entirely unmetered. Meters, so far, have been installed only where it was thought that water was being wasted, as in factories, laundries, livery stables, etc. It should be added that the city intends to instal meters quite generally from now on.

Despite the unsatisfactory meterage conditions described the water department officials have succeeded in decreasing the daily tap consumption from 1,011 gals. in 1911 to 856 gals. in 1913. Tap consumption here indicates the average number of gallons per service connection whether in use or not. This reduction of tap consumption was accomplished by a careful inspection of the use of water by various consumers and the installation of meters where water was wasted in large quantities. Had meters been generally installed this consumption of water per tap would no doubt have been still further reduced.

As stated the capacity of the plant has already been reached and the filters are overtaxed during the summer when sprinkling is heavy. The records show that the daily consumption for the summer of 1913 exceeded 12,000,000 gals. at times, and the maximum pumpage during the period of heaviest sprinkling reached the 18,000,000 gal. rate for three or four hours in the evening.

In the light of the foregoing additional illustration of the increase in water consumption following filtration, we again desire to impress upon filtration engineers the desirability of making due allowance for such increased demand in proportioning filtration plants for cities which formerly have had an inferior water supply. Closely related to this consideration is the desirability of keeping meterage consistently abreast of filtration. A city's meterage program must be considered in connection with its filtration program if the engineer is to make a dependable future consumption estimate.

Many engineers who appreciate the full significance of the foregoing observations may be called upon to design a filtration plant to fit a certain bond issue, being thereby constrained to limit the capacity of the plant in order to keep within prescribed cost limitations. To such we trust the foregoing data will be of some service in convincing city officials of the wisdom of taking into consideration, in due season, the increased demand for water in a city following filtration of the public supply.

Roads and Streets

Some Principles Pertaining to the Administration and Financing of Highway Construction and Maintenance.

In an editorial concerning the technical organization of the road system of France published in our issue of Oct. 22, 1913, the importance of road administration as a feature of state public works management was commented upon in connection with the suggestion that useful lessons in highway administration may be learned by carefully considering the methods employed in countries having such complete and perfected organizations as exist in France. A noteworthy discussion of some of the important principles concerning the administration and financing of highway work in the United States was given in a paper presented before the tenth annual convention of the American Road Builders' Association by Col. E. A. Stevens, Commissioner, Department of Public Roads of New Jersey. The major portion of the paper follows:

The condition of the public roads, which is generally admitted as a good measure of a community's advance in civilization, depends upon two factors: (1) Correct planning, design and construction; and (2) Proper upkeep. Both of these are in turn dependent on efficient administration. In the latter term are to be included all of the numerous details of management, exclusive of the technical engineering work. Without proper administration no extensive system of good roads is possible or even conceivable. It is in this very aspect of the road question that we are most backward. As a pioneer nation we have had to combine many trades in one man. The tradition of those days hangs around us still. We still act on the principle that any two-handed man who can earn a living and is fairly honest can care for a road and that the highway engineer is an almost superfluous luxury whose duty is mainly drawing pretty but useless pictures of work and whose service and pay should be cut down to the irreducible minimum. The engineer—and by this I mean the good and consequently well paid engineer—is essential to good roads; you can't build them and have them stay built without his aid.

Our misapplied inheritance from a glorious past does not limit its baneful influence to the engineer and to his work. In this respect, however, it is losing force every day. We too often spend state money in building magnificent roads without taking any thought of their administration after completion. In New Jersey, and in many other states, we turn over these roads to elective bodies of constantly changing politics and personnel, bodies not fitted by training for the technical work of administration, without requiring the employment of trained men and without setting any required standard of maintenance. At best these bodies are unconnected units and their work is not systematized and coordinated to a common end.

Whatever may have been the case 20 years ago, the motor vehicle and its ever growing commercial importance today demands continuous roads. The condition of every part of such a road is a matter of general concern and not to be localized, as it is, by the neglect of any portion of what must be a continuous whole if the value of the investment is to be realized.

While this statement is true as to all public roads, its importance is greatest in the case of those roads which form the main arteries of road traffic. These will naturally cross municipal and county boundaries. The cost of the upkeep of any portion within one municipality bears no close relation to the benefit to the community nor to its financial ability. At the same time in the case of less important roads the benefit accrues largely to the locality. Hence it becomes advisable to classify roads so as to provide equitably for the distribution of the cost of maintenance. In whatever manner this classification is made and whatever system of administration is adopted, certain underlying principles must be adopted if the best results are to be expected.

(1) There must be a central control to secure the direction of all efforts to a common end; (2) There must be an adequately trained force to carry on the work; (3) Sufficient means must be provided for the work.

As to the first of these principles, whether there be a system of state highways or whether localities are left in charge of their own roads, some one must lay out the through lines and there must be some uniformity in their design and improvement as well as in the condition in which they are to be maintained. These cannot be left to the chance agreement of a number of municipalities. Work both in construction and repair must be planned not only with foresight and intelligence but to give the best results for the state at large and not merely for the convenience and benefit of the locality in which the road is to be built. For the purposes of scientific design statistics as to traffic over various types of roads and the comparative cost of the maintenance under varying conditions of load must be gathered, compiled and analyzed. New types of construction must be tested and proved. All of this is work which local bodies are seldom fitted to perform even were their means and the value to them of the data obtained sufficient to justify the considerable expense involved.

The need of a trained and disciplined force is beyond doubt. How it is to be secured is the question. The most important men will be those at its head and these naturally will be engineers. Highway engineering in its theoretical requirements is simple enough. Except as to bridges few problems arise requiring high mathematical training. There is, however, an almost unlimited field for the application of the thoroughly digested information that comes only with extended experience. The power to acquire, digest and systematically apply this information is what makes the practical engineer, such as is needed in highway work. This training is best acquired on the work itself by young men with thorough technical preparation. The ideal road force must attract such men, must offer them a training and a future. Only in a large force, free from political control and with fair chances for promotion as reward for proven efficiency, can such conditions be insured. What is true for the technically trained young man is likewise true for the repair man. All officials actually in charge of roads, no matter how important or unimportant their tasks, should form parts of a trained force, without consideration of whether they be employed by township, county or state. Too much stress cannot be laid on the need of a trained force. To every state official will, I believe, at once occur many instances of waste by improper repair, of petty graft and of stupid or ill-planned work. Without a force which at least is supposed to be trained, it is hard, if not impossible, to enforce that individual responsibility which must underlie any efficient administration. The roads also are one system. The driver knows no difference when he passes from a state highway to a township road. Good roads are needed under all jurisdictions if the commercial potentiality of any general highway system is to be developed. Hence the coordination and unity not only in the state force but in all officials charged with providing efficient road service for the people of any state. To-day the civil service system seems the only means of recruiting such a force. Whatever may be the shortcomings of this source of supply, it is preferable to political control. Promotion for efficiency in all grades is necessary for the best results.

Above all any force must have a spirit of loyalty, of devotion to its work and of pride therein; in a word an "esprit de corps." With this, men far from ideal in attainment and organization can and will accomplish results impossible to the most perfectly fitted and organized body not animated by this life giving force.

A road program must comprise not only a list of the roads to be built and kept in repair and the organization of a force to do the work, but must consider likewise the amount of money needed and the methods by which it is to be raised, and the coat must be cut to fit the cloth. As it is generally the case that the state shares with communities in the cost of much of the road work, the question of finance is not usually to be solved by merely determining the amounts to be furnished each year by the state. The ability and willingness of the counties and municipalities to bear their share must be weighed. At best this is a hard and unsatisfactory problem, but it must be faced and solved, in each case probably by different means and in different ways as local conditions and laws may require.

For any public body the choice of the bond issue to provide for original improvement or of the "pay as you go" method or some combination must be made. The elements in this choice are the comparative cost to the taxpayer and the return to the public in road service. My own experience is that these are seldom considered. The decision is usually based on the desire of finding the easiest way out of a difficult dilemma, satisfying the demand for good roads and keeping down next year's tax levy. The troubles of the officials in charge 30 years hence rarely worry one very much.

I have worked out an example as follows: Take a case involving $10,000,000 in bonds, issued at the rate of $1,000,000 a year for 10 years, bearing 4 per cent interest and due 30 years from date of issue, with a sinking fund charge of 1.783 per cent; assume that 100 miles of road will be built each year; that the contractor will keep the road in repair for one year after acceptance and that the cost in repair thereafter is $500 a mile. The last bonds will be paid in 40 years. The cost to the community will be:

Sinking fund (construction)	$ 5,527,300
Interest	12,400,000
Upkeep	16,750,000
Total	$34,677,300

or nearly $862,000 a year as an average. The yearly charge varies from $57,830 in the first year to $1,073,800 for the 13th year, at which maximum it remains to the 30th year, after which it decreases to the 40th year, where it becomes $500,000, the repair charge.

If, on the other hand, a fund of $860,000 be set aside each year to keep roads built in repair and the balance used to build roads, the 1,000-mile system would be finished in less than 17 years. The cost of the system with its maintenance to the end of the 40-year period would be $25,779,000. There is a saving of almost $9,000,000 against which the community has enjoyed the use of the whole system for about nine years' more time.

The money spent will have been disbursed as follows:

For construction	$10,000,000
For upkeep (40-year) period	15,779,000
Total	$25,779,000

If the average yearly disbursement of $860,000 be kept up for the whole period of 40 years, it would be possible to increase the total mileage to about 1,525, but from that date on the upkeep charge would be $763,000 instead of $500,000.

All of these plans, as well as one providing for a gradually increasing charge, deserve and should receive consideration before deciding on the method to be followed in any case.

The general consideration of how the roads, after being planned and built, are to be kept in order, is a vital part of administrative work. A thorough comparison of the total cost during a period under different financial plans involves such consideration. The changes of the last few years in the character of road traffic, its unforeseen increase in amount and in destructiveness to road surfaces emphasize the need for the most liberal allowances for this item. There are certain items in the cost of a road that are permanent investments. The right of way is preeminently of this class; the grading, drainage, and, in some cases, the foundation, are of such a nature that with reasonable care they can be kept, at a small yearly charge, in such condition that no depreciation takes place. The surface, however, is frequently of a material that cannot be made good every year. These, after a term of years, will have to be replaced and hence an item of damage that cannot be made good every year will introduce a depreciation charge to further complicate the problem.

The City of Winnipeg, Man., will expend $1,000,000 for improvements to light and power system. Half of this sum will be spent for constructing a new flexible tower transmission line, 77 miles long.

A Suggested Program of National Highway Legislation and Plans for the Economical Expenditure of Federal Funds for Highway Improvement.

The editors have received the following communication relative to a far-sighted federal legislative program on highway building and relative to the equitable and economical expenditure of money appropriated by the federal government for constructing a national system of highways:

Sirs: I am very deeply interested in the highway proposition and am taking the liberty of writing to you in relation to that phase of it which is now before Congress. It seems futile at this time to discuss the advisability of appropriations by the federal government for highway building. It appears inevitable that such appropriations are to be made and such being the case, all who are interested should exert themselves to the utmost towards influencing efficient legislation and the expenditure of the money to be appropriated in a proper manner. I believe it most unwise to make large federal appropriations for highway work until all aspects of the undertaking have been thoroughly examined and the very best and most equitable scheme devised. There is not such need for haste as to necessitate a beginning which may be wrong. Many years have passed since federal money has been extensively used for highway building and although the need for more and better highways is urgent, a delay of two or three years more in order to ensure a proper beginning will do little real injury to anyone and may prevent the doing of that which later on all will wish undone.

For criticism I recommend the following:

1. That Congress request the Secretary of Agriculture to prepare a map of the United States showing a system of national highways; this map upon completion to be forwarded to the President for transmittal to the Governors of the various states with the request that they obtain the legislative approval for the portions of the routes lying within the respective states and return following the close of the next session of the legislature with the approval of the respective state legislatures of it or of a modification. After the closing of all of the aforesaid legislative sessions, but not later than three months previous to the assembling of the following regular session of the U. S. Congress, the President shall forward all of the papers in the case to the Secretary of Agriculture requesting the preparation of a map of the United States showing a system of national highways which shall conform to the actions of the various states, and also a map showing the system of national highways recommended by the Secretary of Agriculture in the light of later information. These two maps shall next be forwarded to Congress in the usual manner and with all the papers in the case. To each of these maps there shall be attached a list of the roads, each approximately 100 miles in length, making up the said national system and each road shall be so designated in the list as to render all easily identified with the roads shown upon the map.

2. That with the aforesaid information before it, Congress shall proceed to adopt a system of national highways, and cause to be published a map showing the adopted system, together with a list of the roads making up said adopted system; this list to consist of roads each approximately 100 miles in length and listed in the order in which they shall be constructed. Until amendment the aforesaid approved system and list of national highways shall be the system upon which all money appropriated by Congress for highway building and maintenance shall be expended, and the list shall determine the order in which building shall proceed.

3. That Congress shall next direct the Secretary of Agriculture to survey and prepare plans, specifications, estimates, proposal forms and contract forms for the first 1,000 miles

of the aforesaid approved system of national highways and report to the following regular session of Congress the amount of money needed to construct the said 1,000 miles of highways. The act of Congress just referred to shall carry a continuing appropriation of $300,000, or as much thereof as may be necessary, to pay the cost of doing the specified work of surveying, etc.

4. That Congress upon the receipt of the report from the Secretary of Agriculture stating the amount of money necessary for the construction of the said 1,000 miles of highways, shall appropriate said sum, make it a continuing appropriation immediately available and direct the Secretary of Agriculture to proceed to construct, in accordance with said plans and specifications, the said mileage and report to the next regular session of Congress the number of miles of highways so constructed and the amount of money needed to properly maintain such mileage during the following fiscal year. This appropriation act shall also direct the Secretary of Agriculture to proceed with the survey and preparation of plans, specifications, estimates, proposal forms and contract forms for the second 1,000 miles of the aforesaid approved system of national highways and report to the following regular session of Congress the amount of money needed to construct said second 1,000 miles of highways; and it shall make a continuing appropriation of $300,000 or as much thereof as may be necessary to pay the cost of said surveys, etc.

5. That Congress at its following regular session and upon receipt of the reports from the Secretary of Agriculture, called for in recommendation No. 4 shall proceed to appropriate the sum specified as necessary to construct the second 1,000 miles and to maintain the portions completed of the first 1,000 miles of highways, direct the Secretary of Agriculture to proceed with said construction and maintenance; and further to pass the legislation which is indicated above as necessary to continue the plant outlined.

The above is an indication of the necessary procedure. It contemplates a system of national highways built entirely by the national government and maintained entirely by the national government. I sincerely believe that if we are to expend federal money it should be upon work which may not be called local. What is needed is construction by the federal government of the trunks: the main branches should be constructed by the states and the smaller ones or feeders by counties and towns. Why should the federal government aid in building local roads? Why, for example, should taxpayers of Massachusetts be taxed to raise money to build a road between Hewitt, Wis., and Easton, Wis.? These taxpayers would, however, be willing to pay their portions of a tax to build a national highway from Augusta, Me., to San Francisco, Calif., or from Chicago, Ill., to New Orleans, La. If local roads are to be built, in what order are they to be selected? I am confident that if once the federal government embarks on what is called the state-aid proposition and appropriations are made without any definite end in view, the result will be something appalling. Suppose the expenditures are confined to post roads; does any one suppose that all of the roads in the United States, nearly 2,500,000 miles, may not become post roads? To engineer the stupendous resulting expenditure there will be necessary a federal organization of enormous proportions, even if formed solely to supervise work done by the states. The carrying out of the program recommended and outlined above will be of something definite.

The method of selecting the system recommended seems nearly ideal; the system is national and the whole nation should have a hand in its determination.

Placing the conduct of the work under the Secretary of Agriculture, and thereby under the Office of Public Roads, is a natural proceeding. Here is an office already organized and equipped for just this class of work. The money being all federal the administration of it should be federal. There would be no gain, but rather a loss in any other arrangement. If a state body is made administrator it would need just as many assistants as would the federal department, and the cost of the work would be more because there will still be necessary a federal organization to provide federal superivsion. Placing the whole responsibility upon the Department of Agriculture insures uniformity in design, construction and maintenance. Such uniformity does not mean that the same treatment would be given to all classes of foundations, or that each problem would not receive the consideration due on account of local conditions. Again, state officers are now in many cases heavily overburdened with their local problems and cannot be expected to administer United States funds with the same regard for efficient results as in the case of funds derived from taxes paid by those to whom they feel more directly responsible.

I believe the scheme outlined above is eminently businesslike and I doubt if in general much fault can be found with it. If the federal government is to furnish money to build highways let it be done right; let the procedure be in accordance with a well considered plan and the carrying out of that plan be solely for the public benefit; let us in this instance build an enduring monument of which we may always be justly proud.

Very truly yours,
Thomas Wateras Warren.
No. 7 De Kalb Ave., White Plains, N. Y.

Methods and Cost of Constructing a Petrolithic Pavement in Cudahy City, California.

Contributed by C. G. Varcoe, Inspecting Superintendent, Los Angeles, Calif.

Cudahy City is a residence and business center developed by the managers of the great Cudahy Ranch, which lies a short distance southeast of Los Angeles, California.

GENERAL CONDITIONS.

Before adopting the method herein described a great deal of time was spent by the executive officers of the management and the writer in investigating various processes in order to ascertain which process would give the most effective paving, with the least cost, for residence streets carrying moderate to light traffic. The process finally adopted consists in mixing asphaltic oil with the native soil and compacting the mixture with Petrolithic tamping rollers to secure a solid carrying base for the streets. The surfacing, it was evident, could be varied to suit needs and cost limits. The tamping method was viewed by the writer with much doubt in the beginning, because of much doubtful and shoddy work he had seen done with the tamping roller. Perhaps because of this doubt, I exercised more care in the oversight of the work. In the outcome I am decidedly convinced that intelligent skillful use of the tamping roller within its proper scope will secure the product of very superior results in many classes of difficult materials.

The soil of Cudahy City is in general a very fine silt-loam. It varies from an extreme of about 60 per cent to 80 per cent of fine drift silt sand to an opposite extreme containing about 85 per cent of gumbo loam mixed with a residue of fine silt, a mixture which is often classed as a straight gumbo. This gave a great variation of material to work with and the pavement on the street which I use as an illustration extended through both extremes with the modifications between.

METHODS AND MATERIALS USED.

For purpose of more exact statement the methods and cost of constructing the pavement on one street, which was the longest stretch of unbroken construction, will be given. It had the more than balancing disadvantage, however, of a double-track electric car line along the middle of the street for the entire length. The length was 3,200 ft., with 15 ft. width of paving on each side of the tracks, totalling 10,666,6+ sq. yds. of paved surface. There were cement curbs on both sides of the street, but no gutters. The paving extended clear to the curbs.

In most of this street the soil was a silt so light and loose that a man would sink to his shoe tops in it when walking across the street. This part required no plowing in the preparatory shaping, but was easy grading. Two separate parts of the roadway, totalling about 750 lineal feet, were dry hard gumbo, that had to be plowed and broke up in large clods, quite difficult to pulverize. The grading was done with Fresno scrapers and a light road-grader. The grading was comparatively light, consisting in taking off an extreme of 12 to 20 ins. at some points and filling similar amounts at other points. The excess material was spread over the lots along the streets.

When the street was shaped water was sprinkled on it by water sprinkling wagons until it was thoroughly dampened to a depth of 6 ins. On a great deal of the work holes had to be first punched with the roller-tamper in order to keep the water from immediately running off the dry powdery silt. The water used throughout the work was obtained at standpipes erected at various points along the streets. As each section was watered, it was cultivated until the proper consistency was obtained and made uniform throughout the 6-in. depth. We found by experiment that we gained the best results when we had the soil well wet before putting on and mixing the oil.

The oil used was one containing about 75 per cent asphalt. It was furnished by the management outside of the contract for the work and is not considered in the cost data herein given, except as regards the mixing and its effect on the work of compacting. The oil came on the work at temperatures varying between 180° and 300° F. It was spread with a Baldwin gravity spreader. Two applications were made, each being varied in quantity between ¾ and 1 gal. per square yard, according to the variations in the soil. The oil was mixed into the soil as quickly after each application as it cooled sufficiently to permit the stock to tramp through it. The mixing was done with a heavy cultivator, followed toward the end of the mixing by a gang disc plow, which turned over the materials to the 6-in. depth. We found it particularly important to get an even uniform mixture of the oil throughout the soil to the depth of the 6 ins., which we had determined to compact. We also found it important to avoid an excess of oil, and to avoid using an oil too heavy to mix readily. I believe that many troubles of earth and oil mixture constructions begin at these points.

Before starting to tamp the mixture, more water was worked in, using as much as the soil would hold without becoming mud. This allowed the feet of the tamping roller to sink in to their full depth and compact a solid bottom. The tamping rollers were immediately followed by a cultivator set shallow so as to keep the surface loose. If this is not done the tamper feet would soon be hitting the same holes, or compacting a mere surface crust which would prevent them reaching the bottom. The water wagons were kept steadily busy spreading water as needed to keep the mixture at the effective consistency. Some sections required water as frequently as twice a day, while other, sections required much less. At least once a day and sometimes twice a day a grader was run over the work to true up the surface and draw in any excess of the mixture that had worked toward the gutters.

After the roadway had been tamped as solid as possible, which is within an inch of the top, the street was well watered and allowed to stand until the surface would not pick up. Then a 5-ton smooth roller (horse drawn) was put on and kept going until the surface was smooth and hard.

The street was then left to set. For this phrase actually describes what took place; the longer it set the harder it got. These light silt soil streets thus mixed and compacted set so hard after standing a month that we found it impossible to make a grader blade cut into the surface when we attempted to smooth out a rough place in one of the streets. We also observed that a 3-ton auto-truck, which was passing back and forth over one of the streets finished to this point of the construction, made no ruts whatever, though the truck was fully loaded one way and partially the other. The heavier soils set more rapidly than the lighter more sandy ones, but the sandy sections finally set harder than the others.

TABLE I.—COST OF LABOR AND RENT FOR TOOLS USED IN THE CONSTRUCTION OF PETROLITHIC PAVEMENT ON SAVILLE ST., CUDAHY CITY, CALIF.

Preparing roadbed, including plowing, scraper work and grader work	$ 81.00
Mixing in oil and tamping roadway:	
2 tampers for 12 days = 1 month of elapsed time for 1 tamper at $35 rental	35.00
1 cultivator for ½ month	2.50
1 disc gang plow for ½ month	2.50
10 head of stock for 12 days, feed and rent	120.00
3 men for 12 days at $2.25 per day	81.00
Total	$241.00
Hauling and applying water throughout work	115.00
Entire total	$437.00
Cost per sq. yd.—	
Preparatory work	$0.0075+
Mixing oil and tamping	0.0225+
Applying water	0.0107+
Total	$0.0407+

It was intended in the original plans to make a surface over this base of ½ to 1 in. of crushed stone and asphaltic oil, but this has been left off at least temporarily. Since the writer left the work, the surface has been finished off with merely a coating of ½-gal. of 90 per cent oil per square yard and covered with fine sand. The sand may make a good smooth finish, but cannot make a very resistant surface; also I doubt the advisability of using so heavy an oil in this combination; it may fail of making a proper bond, because it has no penetrating power, and may thus allow the surfacing to pick up when traffic passes over.

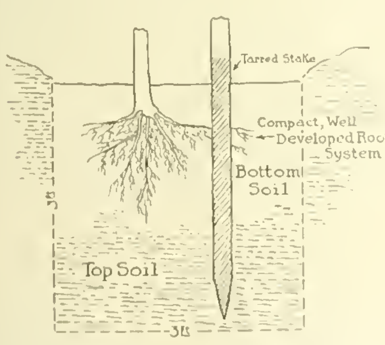


Fig. 1. Method of Planting in Very Good Soil.

The soil worked with in this instance had two marked peculiarities. First the soil was thoroughly dried out by over six months of a Southern California dry season. Secondly the larger mass of the soil is a light silt in its main make-up that lets water run right through it. Consequently we had to use an unusual quantity of water, amounting to a total of as much as 20 gals. per square yard on some of the worse sections. The experience has convinced the writer that most soils can be successfully compacted to perfect reliability for roadway carrying base by the use of similar methods. Too close attention should not be paid to rigid specifications, however. Attention should be given to the character of the soil to be worked, and modifications should be made as may be needed for any great variation in the soil characteristics.

COST DATA.

The figures given in Table I are calculated entirely on a rental basis for both implements and live stock, and on the actual amounts paid to the men. No allowance is made for teams used on the cultivator and on the plow, because such teams were always taken from a roller tamper and the tamper laid off while the teams were so used. But rent is counted for full time on the tampers since it went on whether they were in use or not. The dimensions of the pavement are here repeated for convenience in studying Table I of cost data: Length of street paved, 3,200 ft.; width of pavement 30 ft. (15 ft. on each side of car tracks); total amount of paving 10,666.6+ sq. yds. The costs given do not include the cost of the oil or cost of delivering it on the work.

Street Tree Planting and Cultivation In California.

Methods of planting and caring for street trees are reviewed in a bulletin recently published by the California State Board of Forestry. Portions of this bulletin contain information which is of general utility and some of the most specific of this information is abstracted and given below.

Planting.—It has been found that for best success a hole at least 3 ft. for each of the three dimensions should be excavated. If the nature of the soil does not demand an addition of fertilizer, the top soil is put at the bottom and the lower soil brought to the top. In most cases it will pay best in the end to add some fertilizer. In some cities the street sweepings are saved and used for this purpose. These are quite excellent for lightening the consistency of the soil, but vary considerably in their value from the food standpoint. Again, if the soil is of a very clayey nature, the addition of lime is a valuable point to remember. Figure 1 represents the method of work in a very good soil naturally approaching the ideal. Figure 2 represents the method employed by a local stock company of citizens, which cares for their plantings, in Garvanza, Cal. This method is to be recommended over the preceding in any case. In cases of specially clayey soil this company adopts the method of using lime to free the soil particles shown in Figure 3. It has been found that by the time the roots of the tree reach the lower level that the lime has spent itself and broken up the colloidal clay so that the soil is easily penetrated by the roots. The holes should be prepared before the trees are obtained, so that the soil may have time to settle.

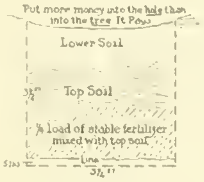

Put more money into the hole than into the tree It Pays
Lower Soil
Top Soil
3½"
½ load of stable fertilizer mixed with top soil
lime
3½"

Fig. 2. Preferable Method of Planting.

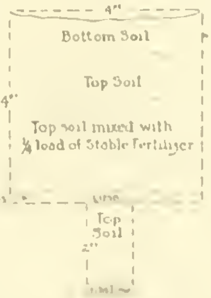


Fig. 3. Method of Planting in Very Clayey Soil.

At all plantings, arrangement is made for watering, Fig. 4. Usually the top 6 ins. of soil is kept loose, so that it can be drawn back to form a rim and the hollow flooded. After the water has disappeared this loose material is thrown back as a mulch. For planting on hillsides the tree is placed as shown by Fig. 5. If, in addition to the dry soil mulch, stable litter or even dry grass can be added, it will be of advantage.

After planting, the tree should be staked, and for obvious reasons the stake should be on the side nearest the source of the prevailing winds. Usually one strong stake, 8 to 10 ft. high, will suffice, and this can usually be removed after the sixth or seventh year.

The tree, as it comes from the nursery, should have a good system of roots with a large proportion of fibrous roots. The main stem should be unbranched, so that it will keep unbranched for at least 12 ft. If this point is assured, there will not be so much pruning needed later to keep the tree in good form. Of course there are some species which will have to be headed even higher, but these cases can be determined in local practice. Since even with the greatest of care some of the root system is lost in transplanting, a corresponding reduction must be made in the top. This top pruning can be well done only by careful study of the individual tree, so that the new growth starting from the undeveloped buds will be able to grow along the most desirable lines.

Some species grow symmetrically by nature and call for little effort on the part of the nurseryman to maintain a good shape. On the other hand, there are a great many species which need considerable care. This is a problem for our nurserymen, and we must demand good materials from him and be willing to pay for the extra care that they must have.

Use only first-class nursery stock and expect to pay for it. Examine your trees; see that they are healthy and free from insect pests; see that they have developed normally, and are not trees which have made the best of bad conditions; see that they have grown vigorously, and have never been stunted or root-bound. Then give them a place to grow which gives them every opportunity for an optimum development, and then do not sit down and await results, but give them constant care, for one can not expect reasonable development with no care at all.

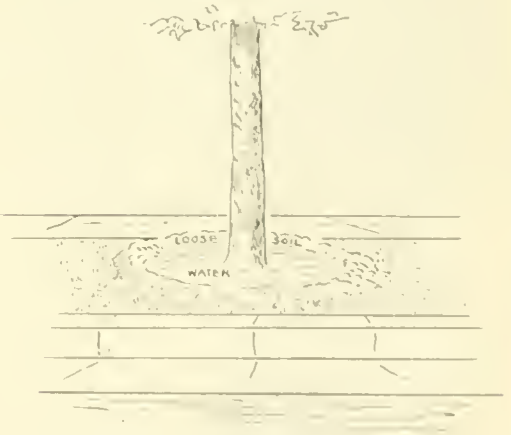


Fig. 4. Method of Arranging for Watering.

The time of planting will vary somewhat according to the locality, the species, and the facilities for care afterwards. From general practice it might be said that it is safe to plant in the spring, after the hardest, coldest rains are over, until the early summer months. Some species are more tolerant and resistant than others, and can be planted all through the summer months. Many more species can be planted all through the summer, and early fall as well, if they are specimens which have been grown in cans or tubs, or have been transplanted so often that a good mass of roots is formed. The advantage of this method is that the work is less of a shock to the tree, and provided a sufficient supply of water can be given afterwards, many of the less tolerant forms can be given this treatment. In using specimens from tubs or cans, care must be exercised that no deformed or root-bound specimens slip into the lot.

The safest generalization is, of course, that planting should be done just as new growth is starting, so that the tree may become established at once and not have the shock of losing foliage and roots which would come if it were moved after growth had started. Nevertheless, if the planter is willing to meet the difficulties, the question of time can be largely disregarded.

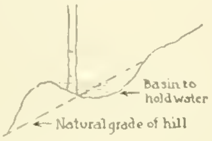


Fig. 5. Arrangement for Watering in Side Hill Planting.

Staking and Guarding.—After planting has been accomplished, a small matter of great importance must not be overlooked. And that matter is the staking of the tree. In spite of all precautions at planting time the ground often settles, or subsequent rains or winds disturb the position of the young tree so that it begins to careen at an angle, and unless promptly corrected this crooked shape is kept for all time. The simplest method for preventing this, and the one which has been proven satisfactory and sufficient, is the single stake. As a general practice stakes from 10 to 15 ft. in length should be used, with 3 ft. below ground. The portion below ground should be tarred, so that it will not decay. If the tarred area is carried up for a foot above ground level the danger is lessened still more. If these are driven in about 10 or 12 ins. from the base of the tree the root will not be damaged, and the tree will be held in place. The usual practice is to tie the tree to the stake at several points with soft hemp rope. If this can be run through rubber hose, so much the better. After eight or ten years the trees have usually outgrown the need of the stake and it can be removed, repointed and retarred and used for other smaller trees.

In many cities it has been found that trees are subject to mutilation by horses especially. There are two methods of combatting this difficulty. The way very commonly used in the East, and in a desultory way by private individuals in California, is to provide a guard of some description. These guards vary from a roll of 2-in. hexagonal wire mesh to a wooden frame, such as is commonly used in Washington, D. C., (Fig. 6) or an elaborate iron guard. For old trees the wire mesh guard is entirely adequate, and for young trees a wire mesh guard, supplemented with stout stakes, will answer all purposes. The wooden guard is a little more effective and is reasonably durable. The only advantage of the iron guard is its durability, and, in some cases, its beauty. Unfortunately, there are many times when it is not a thing of beauty. Of all these, the iron guard is the most durable and the most expensive, the expense in most cases being prohibitive.

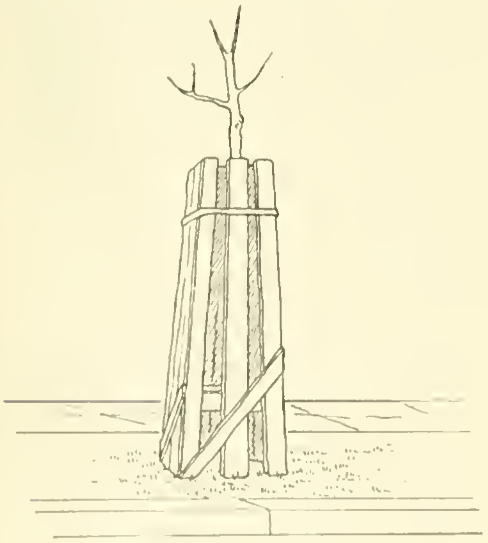


Fig. 6. Satisfactory Type of Wooden Tree Guard.

Instead of incurring this expense, some of our southern California towns have enacted ordinances which prevent hitching to or near a tree, within a certain distance. At first there were a vew violations, but now it is the accepted scheme of things and so far as was noted there are no cases of mutilation of bark or trunks of trees in those towns. Whether this would hold in a great city or not we cannot say. Certainly the intense civic pride of these cities does much to make their respective ordinances effectual.

When we begin to have our trees uniformly throughout our towns, one other feature will rise to importance. That is: the use of grills in the down-town district. A grill is a cast-iron screen of some kind which is put over the soil about the base of a tree to prevent its being packed down by tramping. Every tree must have a certain area of unpaved soil about its base or it can not live and thrive. And in those parts of the town where there is a great amount of walking to and fro, the ground becomes packed down so that it is no longer easily permeated by air and water. In this state the tree can not thrive. It is to prevent this that the grill is useful. These grills are at best an item of considerable expense, but they are a means to an end which is worth far more than any expense at first hand. Figure 7 shows the appearance of a grill from the surface and in section.

Watering.—All trees, of course, receive a thorough watering at the time of their planting, but there must be systematic watering after that. It has been found that it does not pay to leave watering to the individual property owners; it is desirable to have the watering done by the city. If the city takes charge the expense is kept down, and a systematic treatment is assured. During the first three years of the tree's life on the streets it must be watered with the greatest care. After its first watering it is usually watered every two or three weeks during the summer until the winter rains begin. The second year once a month will suffice, and, in some cases, every other month. The third season is usually confined to helping along the backward trees, with water for the others if necessary. Backward trees and trees which show irregular growth must have more frequent and more copious waterings than the normal specimens or more even growing species. Also, trees in situations which are more exposed or which are in soil which does not retain its soil moisture freely, need special attention.

The amount of water to be given to each tree varies according to many factors. Trees after planting need more than established trees; trees with shallow roots need more than deeply rooted trees; trees in very porous soil need more moisture than normal soils, and certain species need more than others.

Fertilization. So far very little has been done in the way of fertilizing and cultivating the street trees. So far as was noted, but one case of artificial fertilization was seen. This case was a planting of acacias on a poor soil which was very rocky. The fertilizer used was barnyard manure mixed with the soil from the 4-ft. hole. Certainly these trees gave no evidence of poverty of soil. Similar instances have been found for individual trees which do not mature rapidly. Chemical fertilizers, chiefly mixtures of acid phosphate and muriate of potash, have been tried in the East with good results, but no records are at hand for their use here. Occasionally street sweepings are saved and incorporated with the soil, but their action is often as much physical as it is chemical.

Cultivation.—As to cultivating—it is the general practice to keep the soil free from weeds and grass about the base of the trees. In most cases an effort is made, with varying success, to keep the soil stirred about the base of the trunk. This insures a dry mulch, and hence a preservation of water and an aeration of the soil, which is greatly to the advantage of the tree.

As has been stated before, in the consideration of the nursery stock, the training and pruning of the tree must begin in its earliest infancy. Since our street is laid out on formal geometrical lines, and its purposes are so strongly utilitarian, some concession must be made to service in the training of a tree. The chief work in the nursery is to insure one single straight leader. There may be strong lateral branches, but there must be one still more vigorous central leader.

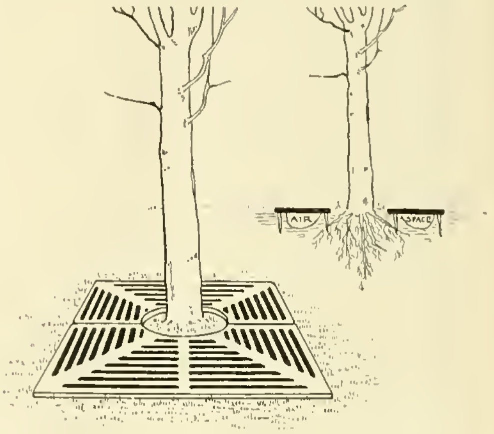


Fig. 7. Grill to Protect Soil at Tree Base.

A Machine for Setting Wagon Tires By Hydraulic Power.

(STAFF ARTICLE.)

Two different methods are in successful use for cold setting iron or steel tires on vehicle wheels. In one method (full circle) the tire is pressed onto the wheel perfectly tight, shortening it in every part of its circumference, while in the other method (edge grip) it is upset or stoved, which shortens it in one place at a time. Tire setting is merely shortening tires until they fit wheels tightly, and it really makes no difference how it is done so that the tire is put on absolutely tight without damage to the tire or to the wheel. A tire may be put on a wheel by either of the two methods of cold setting or by the ordinary hot process, and if the work is properly done, it will stay on as long by one way as another. However, economy in time and expense is usually gained through the use of machinery when the amount of work warrants its installation, and this statement applies to the use of the tire setting machines. Contractors having a considerable number of dump wagons, wheeled scrapers, grading machines, and other vehicles with iron or steel tired wheels, especially if engaged in work in dry, dusty or

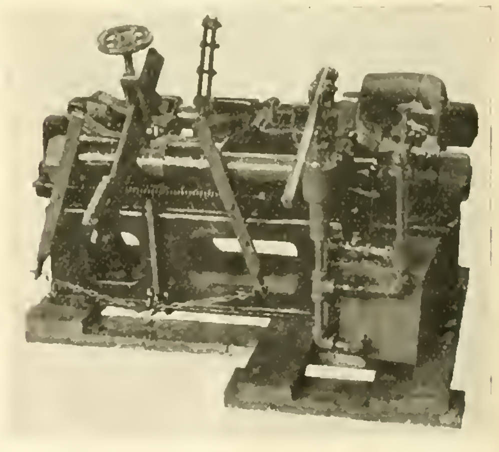


Fig. 1. An Edge Grip Machine for Cold Setting Wagon Tires by Hydraulic Power.

sandy localities where tires loosen frequently and rapidly, would probably find the use of tire setters to be an economy.

The Scientific Tire Setter, shown in Fig. 1, is a machine in which the edge grip principle is applied. It is made with two steel head blocks with upward extending beveled jaws. The head blocks rest on a cast iron base, and to one of them is attached an hydraulic cylinder. Into this cylinder a plunger with a crosshead is fitted. Two adjustable steel rods are placed through the sides of the crosshead and are connected to side lugs on the other head block. Attached to the cylinder is a powerful hydraulic pump, by means of which oil is forced into the cylinder which pushes out the plunger and its crosshead, which causes the two steel head blocks to be pulled together very powerfully. A wheel tire is clamped in the two head blocks by means of beveled tool steel grip blocks which are placed between the beveled jaws of the head blocks. When the head blocks are moved together, as just described, the tire is shortened according to the amount the head blocks are brought together. This distance is governed by the discretion of the operator. Between the grip blocks and the jaws of the head blocks hardened steel liners are placed to adjust the grip blocks to narrow tires. All four grip blocks are made quickly to clamp the tire at one time by the movement of a foot lever, and when a tire is set and the pressure released, the wheel is released automatically. Each machine is furnished with a wheel disher for use in regulating the dish which is especially necessary when a wheel is dished backwards. The machine requires the services of only one man if the proper size machine is used for the average size of tires being set, and the manufacturers state that a 1½x⅝-in. tire can be set every five minutes, and other sizes in a proportionate amount of time. The machines are made in six different sizes ranging in weight from 850 lbs. to 6,000 lbs., crated; and in price from $130 to $550. The machines can also be used for setting tires while hot, provided the proper extra grip blocks for hot metal are used, as heated metal will soon take the temper out of the hardened steel grip blocks intended for cold metal.

The Scientific Tire Setters are made by the Lourie Manufacturing Co., Springfield, Ill., who claim the following advantages for their machines: (1) They are strong, substantial machines in every part and practically indestructible; (2) they work rapidly; (3) they are easily worked, take a small amount of power, and, if the proper size machine is selected, only one man is required to operate it; (4) they are provided with tool steel gripping blocks that are adjusted automatically to conform to the circle of a wheel and grip the tires without marring the felloes; (5) all four gripping blocks can be pushed apart to engage the tire in perfect alinement simultaneously by foot or hand; (6) they can be operated by hand or power; (7) they release the grips instantly. Grips can be set from 2 ins. to 6 ins. apart; (8) the tire can be shortened from ⅛ to ½ in. in one gripping, and as many grips as are needed may be made, one after another on any part of the tire by turning the wheel, working the pump and opening the release valve; and (9) the machines are fitted with the best hydraulic pumps that are adapted to the work.

Buildings

The Problems of the Contractor, with Particular Reference to the Methods Used by the Aberthaw Construction Co., Boston, Mass.

II.

In our issue of Jan. 14, 1913, we gave an abstract of a part of a paper describing the problems of the building contractor. An abstract of the remainder of this paper by Mr. Leonard C. Wason, president of the Aberthaw Construction Co., Boston, Mass., in Vol. LI, No. 5, Journal of Association of Engineering Societies, is herewith given:

SELECTION AND PURCHASE OF MATERIALS

The purchasing department is also busy, beginning before any work is done at the site of the building, and always keeping ahead of construction requirements.

As stated in Part 1, under the head of "The Contract," the contractor is very largely a broker buying materials to be incorporated in a building. Some materials are easy to buy and some require a detailed knowledge of the materials themselves and also of the estimate and the contract and of the relation of one class of material to another. Therefore, there is a distinction made, classifying the simplest articles and duplicate orders as routine buying and others as experienced buying. Of the routine buying, some of it can best be done locally from the job and some from the main office of the company. There has been compiled a complete list of all materials which enter into a job, and then it is decided which of the routine items can most advantageously be bought by the main office and which by the job superintendent, and written instructions are prepared for these items. This list gives in the first column the list of materials, in the second the usual time which it takes to obtain these after placing the order, in the third column some explanatory notes, giving necessary information, and in the fourth the time after the contract is awarded at which information should be received in order to deliver completed building on time. This time assumes that the ordinary four-story mill building will be completed ready for delivery in four months from award of contract. On this list the great majority of items require information within one week, a few in two weeks, and the latest must not be delayed more than a month after the award of the contract, if the probability of delay is to be avoided. It appears strange and engineers have been surprised when we have asked for full-size window details before the footings are in place, but when it is realized that from seven to ten weeks are necessary if the minimum price combined with quality is to be obtained, it will be seen that this request is not unreasonable.

In the purchase of manufactured articles, such as doors and windows, and in the letting of sub-contracts, hundreds of dollars can be saved and better results obtained if handled by a person who by long experience has become familiar with all the details of the materials themselves, who knows the dealers and manufacturers in various localities, and who also is familiar with the relationship of all these materials to one another, as well as with the estimate and contract.

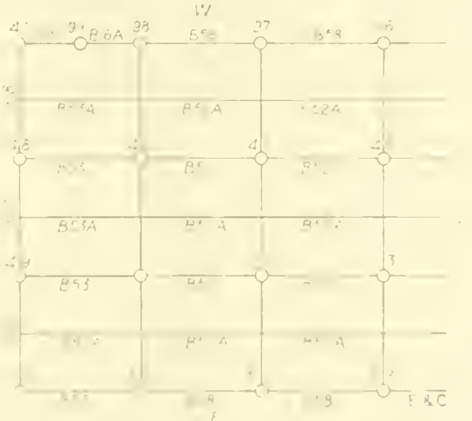


Fig. 2. Part Key Plan to Beam and Column Forms.

Similarly, all changes, either additions or deductions, from the original design, should be handled by one person in order to see that no mistakes are made.

The selection of the aggregate for a job in a new locality is often an important problem and study is put upon it by all of those competent to decide the matter. A decision has to be made before the final design of the mixing plant and its location can be determined, because team and railroad deliveries will be at different places. The purchasing department never buys an unknown sand until it has been thoroughly investigated by a testing laboratory, although this may cause an annoying delay.

The lumber and steel schedules are generally the first to demand attention. Lumber used in contact with cement is almost always ordered planed on four sides. A correct ordering schedule for lumber cannot be made, of course, until the work of detailing the forms is nearly completed in the drafting room. It is possible, however, to make an approximate schedule of the same for use in canvassing for prices. By referring to standards we generally are able to tell in advance what sizes are wanted, and in this way are ready to order material forward immediately after the centering details are completed. As the lumber arrives, the material of different dimensions, qualities and finishes is piled, according to a pre-arranged plan worked out in the office, handy to the woodworking shop, and is given a number or letter obtained from the key plan.

The steel is taken off of the plans by one man, and is checked independently by another. This schedule is then studied in regard to reducing the variety of lengths, and frequently these can be reduced to one fifth of the number shown on the plans. It is usual to order a schedule of bars ¾-in. and larger in multiples of 6 ins. and to buy steel ⅝ in. and under in the longest lengths which can be put on a single flat car, and then cut it up on the job. The largest size recommended for square bars is 1¼ in. and for round bars, 1⅜ in.; while the smallest size is ¼ in., all sizes to be multiples of ⅛ in. On one large job there were over 100 different lengths of bars shown on the plans while by scheduling the large bars in multiples of 6 ins. and ordering small bars in long lengths, as above described, the mill order was reduced to 46 different lengths. This method increased the weight of the order an amount which cost only $18 in excess of the cost of the exact schedule taken from plans. This amount was saved many times over in the greater convenience of sorting and handling the steel. The manufacturers are required to deliver the small bars in bundles weighing from 125 to 145 lbs. This insures that the men get a load of the proper weight, and thus obtain greater efficiency. The bars are also easier to handle and it is not necessary to pick up one bar at a time and be troubled with the unraveling found with small sizes.

MANAGEMENT OF CONSTRUCTION

In the general management of the construction operations the writer's company has made radical changes from customary practice in the erection of reinforced concrete buildings. These changes have been made, not from theoretical considerations and mere office studies, but for the sole purpose of reducing costs and at the same time improving the quality of the workmanship. As a result of the introduction of the methods described below, the labor cost of making, erecting and removing forms—which is one of the largest items of expense in a reinforced concrete job—has been reduced during the past four years over 30 per cent.

The general purpose of the new methods has been to plan the construction details in advance so that each skilled workman, as well as each laborer, will know just what to do, will work to the best advantage without the delays usually considered unavoidable, and will be assigned to work of a nature best suited to his ability. In this way he is enabled to go ahead without the waits and the bother—equally vexatious to the workman and to the bosses —that are inevitable under the ordinary methods of management, no matter how well the work may be handled by the superintendent.

The task has been no small one and is not yet completed. It has involved, as essentials, a thorough study of the plant layout, the establishment of standard methods of construction design and operation, the making of detail plans for forms, the routing of the men and materials, and the establishment of task and bonus arranged to give the workmen an appreciable increase in pay for speedy and accurate performance. It was recognized that Mr. Sanford E. Thompson had carried previous studies along these lines further than any one else, and he was, therefore, engaged in the first instance to co-operate with us in furnishing the basis for the systematic handling of the form drawings and the establishment of the routing and of the task and bonus.

The new methods have paid, both from the standpoint of the contractor and of the owner, permitting smoother operation of the work, and, after making full allowance for increased overhead charges, materially reducing unit costs.

DRAWINGS FOR FORMS, STEEL BENDING AND PLANT.

In erecting a concrete building, one of the earliest and very important parts of the work to need attention is the form work. A study of comparative costs on different buildings indicates that about 35 per cent of the money spent is for labor, over one-third of which is consumed in the making up, erection and stripping of forms. This, combined with the fact that lumber and labor are continually advancing in price, would indicate that a very careful study devoted to reaching the greatest simplicity and regularity in making up panels, and of using the lumber with the most efficiency, is a vital factor in low-cost work.

Forms.—In what follows, by panel is meant several boards cleated together so as to be handled as one piece. It is perhaps needless to state that in order to gain this efficiency, the study of the forms should not be left to carpenters, who are paid to drive nails; nor to the boss carpenter, who has his hands full overseeing the men under him; nor should the superintendent, with the burden of the whole building to carry, be given this problem; but it should be the sole work of a corps of men trained to "think forms" all the time, under the direction of a competent leader, whose experience in different types of buildings shall be considerable and whose judgment is reliable. The principles of design are standardized for beam floors, mushroom floors, columns and walls, and are never changed without a very valid reason.

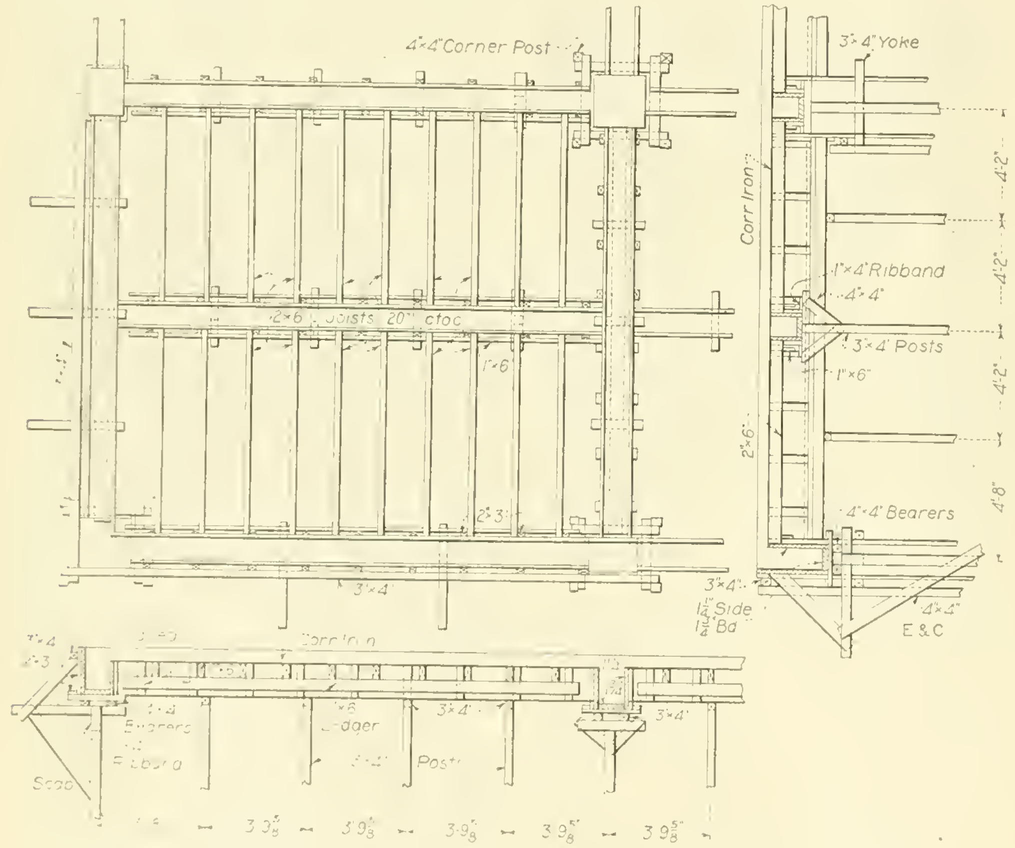


Fig. 3. Assembly Drawing of Centering for Floor Panel with Its Beams.

In order for such a study to reach its greatest usefulness, it is absolutely necessary that the engineer furnish plans, showing clearly beams, columns, slabs, walls, etc., to the contractor before the job is started. Having the complete plans in advance, it is possible to lay out the panels for columns, beams and slabs, not only in regard to the first use, but also to make provision for removing or adding strips where columns or beams are reduced or increased in width, and for cutting off or piecing out where changes of length occur; in fact, to follow every panel from one position to another throughout its usefulness on a job. Further, it is essential that this complete layout should be made far enough in advance of the actual work to enable schedules of the lumber to be made from it, and leave time for a thorough canvass for prices before placing the order. Buying lumber by guess, which necessitates cutting it up without relation to future use, is decidedly wasteful.

A form layout such as mentioned would consist of: First, key plans showing the location of each panel throughout the job; second, assembly plans showing the use and distribution of the lumber and the way all parts go together; and third, the actual panel details. Typical plans are shown in Figs. 2, 3 and 4. Figure 2 shows a part of the key plan for beam and column forms; Fig. 3 shows an assembly drawing of the centering for a floor panel, with its beams; and Fig. 4 shows a typical column and beam panel. Without complete plans of the building it is impossible to make the key plans or to lay out the panels.

A standard method of drawing and marking key plans and assembly and panel details is used, so that the workmen, in changing from one job to another, may find everything as they have been accustomed to it. On the sheets nothing should be left to the outside men to add, but every detail, size of boards, number, location and size of nails, clearances, cleanouts, etc., should be indicated or called for. The kind and dimensions of lumber in one city differ from those in another. The design must be in accordance with the market.

The question next arises, "What is the cost to the contractor of such layouts?" So many variables enter into this that it is impossible at the present time to give any actual standard. It is obvious that, in a job that is simple and regular, several stories in height, with columns and beams that do not vary in size in the different stories, there will be fewer panels to lay out, as each panel will apply to a large number of places.

Actually, on a large regular job this cost is very low, almost insignificant, while on a small job, especially if irregular, with small panels and frequent changes, it may be relatively quite large. On a mushroom job the cost is usually less than on a beam job, as practically all the panels are confined to columns and wall beams, if corrugated iron is used in forming the slabs.

A careful study of the spacing of posts, girts and joists of various dimensions, to carry the varying dead loads of concrete, has been made, and tables prepared based on the net section of dressed lumber, so that designing is reduced to looking up dimensions in tables and laying out results. Thus hereafter the drafting room cost can be somewhat reduced.

On a typical mill building having the lower floor of beam-and-girder construction, the others being mushroom, there were 179,000 sq. ft. of area in contact with the forms. Forty-two drawings were made, detailing 326 different kinds of panels. The number of panels scheduled to make up was 996. This gave an average of three panels of a kind. The maximum number of repetitions of a single panel was 48; the minimum number was 1. The cost of detailing forms amounted to 13 cts. per hundred square feet of contract area. If this building had been mushroom throughout, instead of being partly beam and partly mushroom, the number of panels would have been reduced and the corresponding cost lower. The expenditure of this money is amply justified by the reduction in the total cost of form work.

So far, in showing the importance of getting complete structural plans very early in the job, but one portion of the subject has been touched. Frequently the decision as to what type of window is to be used is delayed. In concrete work a different kind of rebate is formed for each varying form of window. With rolled metal sash a strip rebate is left in the column and wall beam bottom, while for wooden frames a rebate from 1 to 4 ins. deep, depending on whether plank or double-hung box frames are used, must be left extending from near the outside face of the column or beam to the inside face. These details must be in the hands of the contractor before the erection of forms is started, preferably with the other details. All inserts, anchors, wall ties, etc., should be shown in time to enable them to be ordered and attached to the forms without delaying the work. Piping and plumbing must be designed so that sleeve coring holes may be left for their passage. Electric wiring must be arranged for, so that conduits may be laid on the forms before concreting. In fact, plans on a concrete building cannot be too complete, or furnished too early.

Frequently, a contractor desires to change certain details on a building, believing that some other method may give the same results more cheaply. The nature of these details was mentioned under the heading "Analysis of the Design—*Minor Features.*"

Steel Bending.—A sketch is made for every shape and size of bar that is bent on the job, the number, size and location of each style of bend are tabulated, and these data are furnished to the steel foreman.

Drawings for Plant.—The general arrangement, amount and kind of plant are determined for any given job by the writer, general superintendent and job superintendent. These data are turned into the drafting room and laid out to scale. The drafting room has standard plans giving the principal dimensions and all necessary information concerning the use of each big tool such as mixers, elevators, engines, wood-working machinery, temporary buildings, and any other information that is necessary to enable a draftsman without any special knowledge of design to combine the data given him in free-hand sketch form into working drawings for the use of the men on the job.

THE PLANT.

The study of the amount and location of mechanical plant has to be made immediately after the contract is awarded if work is to be started promptly, as part of it is put into operation with the very beginning of construction. As much depends upon the amount, design and promptness of delivery, so far as economical construction goes, the best thought of the whole organization is put on the subject. For instance, one of the younger men made a study for a large job on basis of a story a week. To obtain this rate one mixer and two stories of forms more were required than if the progress was slowed down to a story in nine working days. Such speed was not demanded. A conference of all hands corrected the first study, making an estimated saving in job cost of about $10,000.

Before the location of the concrete mixing plant can be decided upon, the source of supply for the aggregate and cement must be settled, which in turn settles the method of delivery. If materials are delivered by teams, the mixing plant will usually have to be on the front of the building for the economical handling of the aggregates; if by rail, on the rear of the building. The size of the mixing plant is determined by the amount of concrete to be mixed for the entire job, the time allowed by contract to complete the mixing, and the maximum quantity that has to be placed in a single day. Usually this can be done by one mixer taking as a charge a barrel of cement and the aggregates in any proportion. A common arrangement for the concrete mixing plant is to lay an industrial railway track so that the aggregates are unloaded by shoveling from steam railroads into industrial cars which dump directly into a super-hopper, set flush with the ground, over the mixer. A pit is dug outside the building sufficiently deep to take the super-hopper, feeding the mixer by gravity, and the mixer in turn dumps by gravity into an elevator bucket that runs vertically through a steel tower just outside the wall, so that it discharges through a chute into the building. This, as well as all other tools, when possible, is driven by electric power. The pit acts as a sump to drain the surrounding ground and has to be pumped out.

The method of distribution of the concrete through the building requires some consideration. If a building is compact, and large masses are placed within a limited area, spouting is desirable. If the building is spread out and quite small quantities of concrete are used, two-wheeled push carts are used, while if large quantities are required over quite an area, industrial tracks laid on horses so as to elevate them about 18 ins. above the forms are run over the latter to distribute concrete a batch at a time. The amount of track, the number of curves and switches, the number of cars and all the minor items down to the last bolt must be carefully determined at the very outset in order to save delay in erection and several shipments in less than carload lots.

If it is decided to set up a wood-working mill, the kind and amount of machinery necessary must be carefully determined according to local conditions. If, in the shop, only bench and cut-off saws are used. On the standard sketches on file in the drafting room all the rest of the information can be found to make it a complete plant. If, in addition, a planer, boring machine and emery wheel are wanted, it is only necessary to look up other standard drawings to get the correct size of shop and the arrangement of tools, shafting, motor, benches, etc., which previous experience has indicated to be necessary for best results. The wrong location of mill and the wrong machinery sometimes cost more than the possible saving by the use of power tools over hand work, and these questions must be carefully considered and settled at a very early stage.

The question of temporary buildings often is serious. The size and location for the temporary office is a simple matter, but for cement, tools and equipment more thought is needed. Sometimes a cement mill is so regular in its shipments and the railway service so sure that a small supply only need be carried on hand for emergencies. On other occasions it may be necessary to store sometimes 20 carloads. It is not easy to determine these factors, which are beyond our control, without the actual experience, but unless a fairly close approximation to the actual facts is worked out there is likely to be extra cost from too big a building unused, or too small a one to receive the materials to be cared for, which increases the demurrage account.

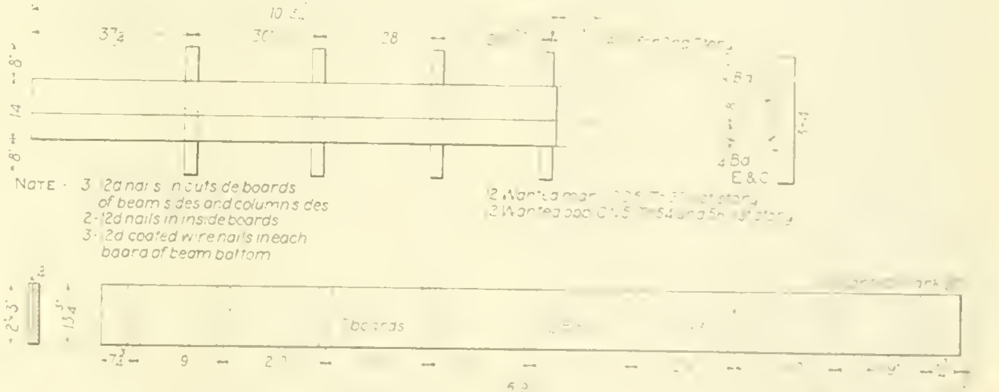

10
37½
30
28
8"
14
8"
NOTE - 3 12d nails in outside boards of beam sides and columns sides
2-12d nails in inside boards
3-12d coated wire nails in each board of beam bottom
E&C
boards
7½
9
63

Fig. 4. Typical Column and Beam Panels.

There are standard lists of small tools compiled for quick use for jobs of various sizes and characters, so that it is merely necessary to call for schedule No. 1, No. 6, or whatever it may be, to start work.

The method and equipment for handling excavation must be settled at the earliest stage in the discussion of plant.

Staging.—A frequent cause of injury to workmen is the breaking of stagings on which they are at work, and the writer believes that this is due to leaving the design and execution to the workmen themselves. Although there may be a large quantity of excellent material at hand, it is not uncommon for workmen to pick out inferior stock for building a stage. To avoid these difficulties the principle has been adopted of designing all stagings in the office for any important work, as for the outside of a building. The stage is laid out, giving dimensions and spacing of lumber, the sizes, number and location of nails, whether the stage be built from the ground up, whether it be a cantilever from an upper story window, or a swing stage hung by ropes from the roof. The plan states the maximum safe load which the stage is permitted to carry. Material is selected for the workmen, and before any are allowed to use the staging its construction is inspected either by the superintendent or by his authorized representative. In this way as much safety is insured the men as the nature of the undertakings will permit. The men work with assurance of safety and therefore ultimate economy results.

ORGANIZATION OF JOB FORCE.

After the job superintendent is determined upon, he is taken into consultation for every other step which follows. The selection of all his subordinates is made in co-operation with him—assistant superintendent, clerical force, carpenter, labor, concrete, steel and other bosses; the principal mechanics and even the leading laborers are all selected in advance from among those working on other jobs. The superintendent, where possible, is given at least a week to study plans and specifications in co-operation with his principal assistants and the office force, before anybody appears at the site of the work except for the mere purpose of studying local conditions. By this time a well-defined plan has been worked out as to the sequence of various operations, the kind and amount of mechanical plant, and its location; schedules of the materials first needed have been made and orders placed, so that by the time actual operations begin in the field the plans are so matured that there are few false moves. The peak of the superintendent's load is the first few weeks during the organization and starting the work. During this period he should have his full executive force, although each subordinate may not at that time be doing nearly as much as he will be called upon to do later.

PLANNING, ROUTING AND COST ACCOUNTING.

During the early stage of actual construction, the superintendent makes a careful study of the handling of the work from the beginning to the very end, making a detailed schedule of work along the general lines which had been previously determined in the office.

The job office work is in charge of a chief clerk, who in addition to overseeing the work of the material clerk and time-keeper, has also charge of the cost-keeping. The chief clerk figures from the time sheets the cost, and two records are kept of these; one for the superintendent's use, in card index form which covers every single item on the job. The other considers only the large items, such as concreting floors and columns, placing reinforcing steel and form work for floors, columns, beams, etc. The latter costs are recorded on cross-section paper. These diagrams are posted where the man who has full charge of each item can consult them when convenient for him.

It has been found that the foremen take a particular interest in these diagrams, as it is natural to suppose they would. Figures which mean a little larger or smaller cost convey some idea to them, of course, but a line traveling upward or downward and above or below a red line which is recorded as the desired cost line enables the foremen to grasp the meaning very easily. Keeping the foremen keenly awake to the cost of the work they are doing has not been, so far as can be seen, detrimental to their turning out good work. It should always be accompanied, however, by more rigid inspection in the first instant, and explained so that they may definitely understand that any lowering of cost shall be considered detrimental to their interest, unless the quality remains equal to the standard.

The execution of a job is divided into two departments: the planning and routing, and the operating departments, the former being in charge of a man conversant enough with our standard methods to plan carefully the different steps of the work and say what to do, while the latter is placed in charge of a practical man of experience, whose duty it is to tell how the work should be performed. Work carried on in this way requires the men to receive orders from two foremen. There is, however, very little trouble encountered by reason of this, as the routing foreman remains in a field office and gives instructions wholly on written slips, and instructs only as to what shall be done. The operating foreman has direct charge of the handling of the labor gangs to make sure that what is being done is performed in the proper manner, both as regards safety, appearance and economy.

The moving boss and his gang are a very important factor in the success of routing. He must be a painstaking, tireless fellow, who is always on the move and must have sufficient intelligence to see that the men under his charge have done exactly as told, taking the correct materials, carrying them to the proper place and laying them right end to and right side up, as per the written instructions from the routing foreman. The men under his direction must be active, strong and energetic—the pick of the laborers, because they do a great deal of moving about, and are in small groups or alone; therefore they are not under the close eye of their foreman. They must be men who will work without being too closely watched. The moving boss receives instruction as to how each laborer shall do his work from the operating foreman.

It is necessary for the routing department to be fully posted at all times as to just what is being done in regard to the delivery of material needed and the completion of the tasks specified. This is taken care of by a daily conference between the job superintendent, operating foremen, and routing foremen. In fact, the success of this system depends very much on a complete co-operation between the operating and the routing foremen.

The operating foreman, by mingling with the workmen at all times, is able to give considerable information to the routing foreman for his use in future planning. He has no authority to give instructions regarding what shall be done to any of the sub-foremen on the job, but makes all suggestions to the routing foreman.

The form work, in the older method of management, was the most difficult to organize and systematize with any efficiency, and therefore the first attention was given it under the system of planning and routing. It was found that this class of work was very easy to systematize and with much effectiveness, more so than the handling of concrete or of steel reinforcement, although these, to a less degree, have been brought into the system. The application of this method to form work is as follows: From the piles which are made when the lumber is first received from the dealer, the moving boss moves the lumber to the sawmill in accordance with written instructions. The mill-man is told by his instruction slip just what length to cut these pieces and what ripping is to be done on each board or stick, if any. After this is done the moving boss moves same to the different making-up benches, still in accordance with written slip to that effect. The work is planned with considerable care quite a way ahead so that each making-up bench has at all times the proper amount and kind of boards, cleats, nails, etc., which will be required to make up the panels which each pair of carpenters is told to do by the instruction slip and accompanying drawings. As soon as these panels are made they are marked by proper panel numbers, taken from the key plan, and the moving boss removes them to a stock pile where they are stacked by a pre-arranged method, from which they are taken as needed during erection.

The foregoing method of making up forms has been followed for a number of years with considerable success and improvement over all other methods. Considerable saving has not only been made in the unit price of making up forms, but a large saving in the use of lumber has also been effected. Under this system the carpenter foreman is relieved of the work of laying out each change of panel, as well as laying out the panel details themselves, and, instead of spending a lot of time working on plans, while his men do no work, can give his entire attention to the method of making up the panels and to their erection in the building. It is along these lines that he has been particularly trained by experience, and he is able more fully to give the carpenters the benefit of this experience than he could in any other way. The high price carpenters are also given an opportunity to devote all their time to actual carpenter work. They do not have to bother with looking up the stock which they will require for the panels they are making or erecting, the material being always piled right behind them ready for their immediate use.

The method of task and bonus pay is frequently followed on work, and by means of data previously obtained it is possible to ascertain very accurately what the correct best time should be for the making up of any particular panels by a pair of good carpenters. For doing the work in this tasked time an increase over their usual hourly wage is paid of from 25 to 30 per cent. On the erection of the forms a similar method is followed. All material is moved from the stock piles to the point of erection by common laborers, under the leadership of an efficient moving foreman. The foreman receives all instructions on a written slip. The planning department is able at all times to make sure by so doing that the carpenters have the material they want, when they want it, and where it will be most convenient for their use. No carpenters are allowed to carry lumber, but are restricted to doing real carpenter work.

On the erection of the posts, girts and joints, to support the floor panel forms, instead of erecting each post separately and then putting the girts on top of these, they are assembled as frames in a horizontal position and erected in large units, much in the same way that a frame house is erected. The work is done by one carpenter with the help of three good laborers.

The work which the steel and concrete gangs do is planned by the routing department, but not in quite so much detail as is the case with the form work. The steel foreman bends the bars in accordance with data copied on a card from the office schedule. These data are made out in the office by a clerk, directly from the plans. After bending, the foreman tags the bars properly, stacks them as per the plan, and the card is returned, with the time required to do this work noted thereon. These cards give data under many different conditions, which enables the superintendent to figure time for task and bonus work.

The saving which is effected in the concrete work is due to the fact that this planning of the whole job gives in advance fairly accurate information as to just what will be required of the concrete gang each day; and by canvassing the whole job for miscellaneous work which this gang can do when not concreting, its efficiency is increased quite materially.

From a study of what precedes, it will be seen that, if every class of labor obeys orders and there is an ample supply of materials, with no lack of information as to requirements for future work, there can be gotten a smoothly running, clocklike system, resulting in greater economy than can be obtained under any other method. This system requires about one boss to not over six or eight men, yet enough work can be obtained from these to more than make up for the unproductiveness of the bosses not working with their hands. A slip in any action, or lack of information from the engineers, non-receipt of materials, or trouble with the labor, is disastrous to the successful operation of the system. The co-operation of the engineer is, therefore, most earnestly desired to give full information long in advance of its actual use, in order to prevent any break in the smooth operation of the system.

In spite of the fact that the wages of carpenters have nearly doubled in 20 years, while the quality and intelligence of those who do the work is less, because house and finish carpenters will not do this kind of work; and in spite also of the great advance in the price of lumber, combined with a falling off in quality, the unit costs obtained by the above system are materially lower than when the job superintendent was left to himself. The total gain in efficiency is about fourfold.

WEATHER CONDITIONS.

Protection of Work from Rain.—It is unnecessary to protect form work from the rain, as usually it is only possible to get green lumber, its tendency being to shrink out of shape rather than swell. Rough concrete never need be protected from the rain before setting, only from the concentrated run-off over some portion of the work, which might wash the cement out of a small area. A troweled finish, however, must be protected against being pitted by the rain or damaged by concentrated streams. This is usually done by tarpaulin, which consists of 12-oz. cotton duck, rendered waterproof by being saturated with "Preservo" after being made into covers 20 ft. square. These are supported by a temporary frame. They are somewhat of an inconvenience to the finishers, and decrease the amount of work which they can do. If rain is expected, it is usual to delay work until after the storm is over, if possible; otherwise, to build a shelter. Sudden showers require quick decision as to whether there is time to build a shelter and also as to its cost, as compared to making repairs on the surface if left exposed. As a rule the shelter is built.

Protection from Sun.—If there is to be an unavoidable delay in placing the concrete after the forms are erected, it is best to protect them from the sun if possible, because they shrink so as to open up cracks which permit the mortar to escape, leaving a bad surface on the concrete. Rough concrete does not require any protection from the sun, with rare exceptions. It is usual to cover up a granolithic finish for at least five days. The materials most available and most economically used for this purpose are rough bags, such as come around bales of cotton, or wool, shavings, sawdust, and sand. The object of these is to retain the water which is put upon the surface, and to maintain it at a uniformly lower temperature than this direct sun action permits.

The writer considers it unwise to spray bare concrete in the hot sun, and the use of roofing paper is but little better, because it retains moisture but a very short time. The water falling on a surface heated directly by the sun chills it suddenly, causing it to shrink, and may cause the very cracks which one is trying to avoid.

Protection from Frost.—The aggregates must be free from frost when mixed. A live steam pipe can be shoved into the sand pile, the escaping steam heating it and removing all frost. The same may be done with the stone, but a canvas should be thrown over the top of the pile to retain the heat, which more readily escapes. Where a considerable amount of heating is provided for in advance, steam pipes are laid on the ground, and stone, as received, is dumped upon them. A canvas then is thrown over the pile to prevent storms from getting into it, and to retain the heat. The frost is thus easily and economically removed.

Salt is frequently used in the water to lower its freezing point. It is seldom worth while to heat the water itself. Little care need be used to prevent mass concrete from freezing, as the frost will usually only strike to a depth of about 1 in. Buildings are inclosed with tarpaulin tied to an outside staging, and the enclosed space is heated with salamanders burning coke. Sometimes it is possible to use steam.

Frost is removed from form work by the use of salt and steam, and if the concrete surface is left rough it is common to sprinkle the top surface with salt to prevent freezing. In winter it is very common to put the finish on as a separate operation, after the building is enclosed, and not as an integral part of the construction, on account of the danger of freezing. When it is put on, in this manner, it must be kept from freezing for the first 48 hours.

The expense of providing a protection against the weather is not very great, and good results so surely can be guaranteed that it is not usual for the writer to discontinue work on account of cold weather. Occasionally it is necessary to make some provision for the protection of the men. Shelters or wind-shields are built in front of the benches where carpenters are making up forms, as well as around the men at the concrete mixer, whose work does not necessarily keep them warm. Where excavation is going on in the open it is expedient to have a building with a good fire where the men can warm themselves when necessary; and on a few jobs it has proved to be wise to furnish hot coffee free in these shelters.

Fluctuations in Price of Some Steel Products for the Period 1898-1913.

It is interesting to note the fluctuations in price of some of the steel products commonly used in engineering practice. The monthly prices given in Table 1, which were taken from the Iron Age, cover the period 1898-1913. These prices are Pittsburgh quotations for materials purchased in large quantities, and they will be found of value to engineers in making estimates of cost based on structures which have already been built.

The period covered in Table 1 begins in 1898, the year in which the depression following the panic of 1893 reached its culmination in its disastrous effect upon steel prices. In 1899 prices rapidly advanced as there was developed a demand which the capacity of the country could not supply. It will be noted that prices rapidly declined in 1900, but they were steadier in 1901. They increased slightly at the beginning of 1902, and maintained a constant price until they suffered a sharp decline in the latter part of 1903. Prices changed only slightly until the latter part of 1904, when they advanced somewhat. Further advances were made during the former and latter parts of 1905. Market conditions were comparatively stable for the first half of 1906, after which there was an advance in prices until the levels of 1902 were practically reached. There was little change in 1907 and in the first half of 1908, after which prices receded. The co-operative movement of steel manufacturers checked the decline during the depression of 1908. In the spring of 1909 there were some sharp declines as the manufacturers, particularly of bars, insisted that prices were being kept too high. These cuts stimulated buying, which resulted in upward prices during the latter half of 1909. Prices declined somewhat in 1910, and continued to decline during 1911. Early in 1912 there was a slight decline due to active competition. This was followed by gradually rising prices until the spring of 1913. The high prices paid for early deliveries marked the first four months of 1913. Business fell off during the summer, and in the last four months there were decided declines in prices.

The curves of Fig. 1, which are based on the prices given in Table 1, clearly show the fluctuations in prices, per ton at Pittsburgh, during the period 1898-1913, for steel beams, bars, rails and wire nails. It will be noted that the price of steel rails has not changed since April, 1901.

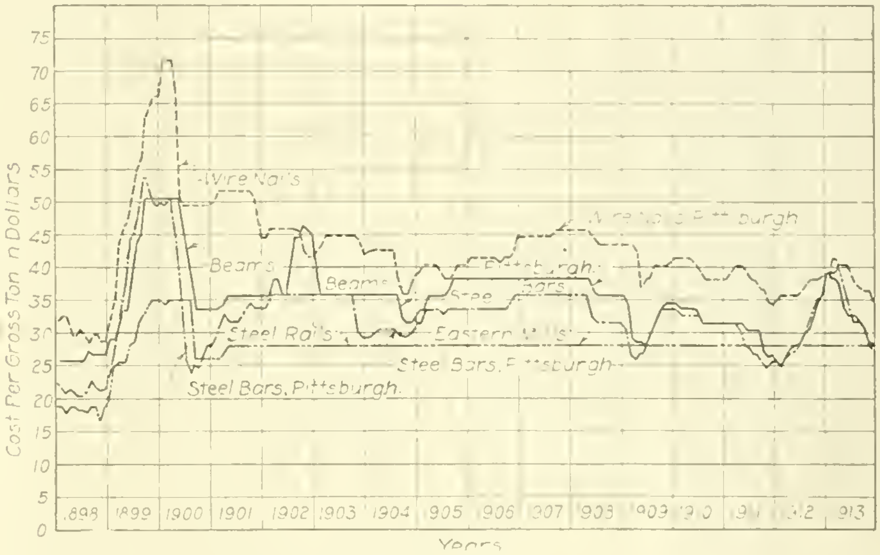


Fig. 1. Curves Showing Fluctuations in Prices Per Ton of Some Steel Products for the Period of 1898-1913.

TABLE 1.—MONTHLY PITTSBURGH PRICES OF SEVERAL STEEL PRODUCTS COVERING THE PERIOD 1898-1913.

	Year.															
Month—	1898.	1899.	1900.	1901.	1902.	1903.	1904.	1905.	1906.	1907.	1908.	1909.	1910.	1911.	1912.	1913.
Soft Steel Bars at Pittsburgh, in Cents per Pound.																
January	0.99	1.02	2.22	1.25	1.50	1.60	1.30	1.40	1.50	1.60	1.60	1.40	1.50	1.40	1.15	1.70
February	0.95	1.12	2.21	1.30	1.51	1.60	1.30	1.49	1.50	1.60	1.60	1.35	1.50	1.40	1.12	1.70
March	0.93	1.36	2.25	1.40	1.60	1.60	1.33	1.50	1.50	1.60	1.60	1.20	1.45	1.40	1.10	1.85
April	0.95	1.56	2.10	1.47	1.60	1.60	1.35	1.50	1.50	1.60	1.60	1.15	1.45	1.40	1.16	1.84
May	0.92	1.67	1.91	1.41	1.60	1.60	1.35	1.50	1.50	1.60	1.60	1.19	1.45	1.37	1.20	1.50
June	0.90	1.98	1.52	1.40	1.60	1.60	1.35	1.46	1.50	1.60	1.45	1.20	1.45	1.25	1.20	1.60
July	0.91	2.02	1.19	1.40	1.60	1.60	1.35	1.50	1.50	1.60	1.40	1.27	1.45	1.23	1.25	1.50
August	0.94	2.14	1.05	1.44	1.60	1.60	1.35	1.50	1.50	1.60	1.40	1.32	1.40	1.20	1.30	1.40
September	1.02	2.40	1.12	1.50	1.60	1.60	1.31	1.50	1.50	1.60	1.40	1.39	1.40	1.19	1.37	1.40
October	0.97	2.39	1.09	1.53	1.60	1.60	1.30	1.50	1.50	1.60	1.40	1.51	1.40	1.12	1.45	1.39
November	0.94	2.24	1.18	1.50	1.60	1.37	1.31	1.50	1.54	1.60	1.40	1.50	1.40	1.08	1.55	1.29
December	0.95	2.20	1.25	1.50	1.60	1.30	1.34	1.50	1.60	1.60	1.40	1.50	1.40	1.12	1.60	1.21
Beams at Pittsburgh, in Cents per Pound.																
January	1.15	1.30	2.25	1.50	1.60	1.80	1.60	1.50	1.70	1.70	1.70	1.60	1.55	1.40	1.15	1.75
February	1.15	1.30	2.25	1.50	1.60	1.60	1.60	1.50	1.70	1.70	1.70	1.52	[illegible]	1.40	1.11	1.71
March	1.15	1.40	2.25	1.52	1.70	1.60	1.60	1.60	1.70	1.70	1.70	1.50	1.50	1.40	1.15	1.70
April	1.15	1.50	2.25	1.60	1.70	1.60	1.60	1.60	1.70	1.70	1.70	1.27	1.50	1.40	1.21	1.68
May	1.15	1.50	2.25	1.60	1.60	1.60	1.60	1.60	1.70	1.70	1.70	1.27	1.50	1.39	1.25	1.50
June	1.15	1.70	2.07	1.60	1.60	1.60	1.60	1.60	1.70	1.70	1.62	1.25	1.48	1.35	1.25	1.45
July	1.15	1.94	1.90	1.60	1.84	1.60	1.60	1.60	1.70	1.70	1.60	1.33	1.41	1.35	1.30	1.45
August	1.21	2.05	1.74	1.60	2.00	1.60	1.60	1.63	1.70	1.70	1.60	1.40	1.40	1.35	1.35	1.45
September	1.20	2.25	1.50	1.60	2.00	1.60	1.44	1.70	1.70	1.70	1.60	1.46	1.40	1.31	1.42	1.41
October	1.20	2.25	1.50	1.60	2.07	1.60	1.40	1.70	1.70	1.70	1.60	1.50	1.40	1.21	1.48	1.37
November	1.20	2.25	1.50	1.60	2.15	1.60	1.40	1.70	1.70	1.70	1.60	1.54	1.40	1.13	1.57	1.29
December	1.20	2.25	1.50	1.60	2.00	1.60	1.44	1.70	1.70	1.70	1.60	1.55	1.40	1.15	1.60	1.25
Wire Nails at Pittsburgh, per Keg of 100 Pounds.																
January	$1.42	$1.43	$3.20	$2.22	$1.99	$1.89	$1.89	$1.75	$1.85	$2.00	$2.05	$1.95	$1.85	$1.71	$1.57	$1.75
February	1.45	1.57	3.20	2.30	2.05	1.92	1.90	1.80	1.85	2.00	2.05	1.95	1.85	1.75	1.60	1.75
March	1.43	1.94	3.20	2.30	2.05	2.00	1.91	1.89	1.85	2.00	2.05	1.95	1.85	1.79	1.60	1.76
April	1.31	2.05	2.95	[illegible]	2.05	2.00	1.90	1.80	1.85	2.00	2.05	1.87	1.85	1.80	1.60	1.80
May	1.31	2.10	2.20	2.30	2.05	2.00	1.90	1.80	1.85	2.00	2.05	1.65	1.82	1.80	1.60	1.80
June	1.35	2.30	2.20	2.30	2.05	2.00	1.90	1.74	1.85	2.00	1.97	1.70	1.80	1.75	1.60	1.80
July	1.31	2.42	2.20	2.30	2.05	2.00	1.89	1.70	1.84	2.00	1.95	1.72	1.75	1.70	1.62	1.70
August	1.26	2.50	2.20	2.30	2.05	2.00	1.71	1.70	1.82	2.00	1.95	1.80	1.70	1.69	1.66	1.65
September	1.32	2.76	2.20	2.30	2.03	2.00	1.60	1.74	1.86	2.05	1.95	1.80	1.70	1.65	1.70	1.65
October	1.33	2.87	2.20	2.28	1.89	2.00	1.60	1.80	1.85	2.05	1.95	1.80	1.70	1.64	1.70	1.63
November	1.28	2.95	2.20	2.17	1.85	1.97	1.62	1.80	1.88	2.05	1.95	1.80	1.70	1.55	1.70	1.59
December	1.27	2.95	2.20	1.99	1.85	1.87	1.73	1.80	2.00	2.05	1.95	1.85	1.70	1.53	1.72	1.55

Building Inspection During Construction.

Careful inspection of buildings during their erection will do much toward insuring good work, provided the inspector is a man who can point out the defects in such a way as not to create antagonism. To secure the best results an inspector must have not only an intimate knowledge of the work but must also possess tact. Some parts of the work are invariably well done while others are often slighted, and it requires considerable experience for the inspector to learn where to concentrate his efforts. The following instructions governing the inspection of buildings under construction were taken from a paper by Mr. Martin Schreiber, in "Aera," and are the result of conferences of the engineers for the Public Service Railway Co. of New Jersey:

(1) Carefully study all plans, elevations or other drawings; also all specifications, and see that the drawings are in conformity with each other and with the specifications.

(2) Check all figures and dimensions on plans and details; check shop details as they are received to see that they do not conflict with scale drawings. Check all erection diagrams in order to recommend some other procedure in an emergency, should the necessity occur. Also see that the specifications, or special intentions, agree.

(3) Keep on the work, in an accessible location, special copies of all drawings, details and specifications, and carefully note on same any changes from original plans, so as to keep a complete record of the work as constructed.

(4) Familiarize yourself with all local conditions or ordinances affecting the work and see that same are rigidly adhered to.

(5) Make the acquaintance and keep in touch with the General Contracter and all sub-contractors or other principals engaged on the work.

(6) Establish diplomatic relations with adjoining property owners, or residents, and see that all their rights are respected by the Contractors.

(7) Acquaint yourself with all traffic arrangements, whether regular or special, that may interfere in any way with the progress of the work.

(8) Secure full information concerning the conditions of steelwork being fabricated in the mills or shops, also of any other materials that may be under the course of construction, and ascertain the names of all contractors, dates of orders and the probable date of shipment, and,

if necessary, closely follow up same in order to secure proper delivery.

(9) Should any incomplete work be fabricated that will need attention and correction at the site, carefully look out for and see that all corrections are made before same is incorporated in the structure.

(10) Keep a record of the arrival of all materials. The Contractor's record should be sufficient, if suitable; but should be carefully checked. Strive to anticipate any shortage of material and use all available facilities to hasten delivery; also arrange for the proper storage of same on the site.

(11) Try to anticipate the need of detail drawings and notify the Engineer or Architect regarding same.

(12) Study the progress of the work and try to determine whether it will be completed in the specified time, and, if in your opinion it will not be finished as specified, endeavor to secure such additions to force and equipment as to secure completion within the allotted time.

(13) See that the Company's photographer is informed as to the progress of the work, and that "progress photographs" are taken at least every ten days.

(14) Keep a daily record of the force employed and the distribution of labor in such a manner that will clearly show the progress of the work, and also of all equipment in use, at all times.

(15) When "unit prices" are called for, make a careful report of the cost of all material and labor which are included in same, separate from each other.

(16) Make careful preparations so that the erection may proceed rapidly and that the period of any disturbance or delay may be reduced to a minimum.

(17) See that all piping or sleeves for same are built through walls and that all other piping conduits, etc., are in place before any floors are laid or masonry built; also see that all chases or recesses are left in walls to receive piping, etc.

(18) Keep in close touch with the Engineer or Architect and consult with them frequently as to the interpretation of all drawings or specifications.

(19) Exercise a constant supervision of any sheet piling, underpinning, or forms for concrete, or any other falsework or temporary structure, and see that same is properly maintained, and not removed until all masonry is set.

(20) Carefully observe surface and sub-surface conditions, and see that all excavations are shored or sheet-piled where necessary. If surface drains are needed, see that same are installed in advance of masonry. Be sure that all waterproofing is thoroughly jointed and lapped. Do not allow back-filling against green walls, unless same are braced; also see that all back-fill is well puddled.

(21) See that all machinery and boiler foundations are built true to detail. See that all stone or terra cotta work is perfect before same is placed in the work; also see that all brick or other mason work is laid and anchored as specified. Be careful and see that all concrete work, whether reinforced or otherwise, is constructed as specified in all particulars.

(22) See that all furring is in place and firmly secured before work is started and that sufficient time is given to dry out between coats, and that a proper backing is prepared for all tile and marble work.

(23) See that all work is carefully aligned and accurately fitted. Closely watch all riveting and see that same is tight; also closely watch the painting of all work.

(24) Carefully examine all work before same is installed; see that all framing is accurately made and anchored and all roughing and furring is built in. All finished work should be carefully inspected as to design, finish, etc.; if necessary, make "mill inspection"; trim must not be applied until plastering is dry. See that all hardware is properly attached, including that to metal-covered work.

(25) See that all flashings are carefully built in.

(26) See that the grades of pipes are uniform, properly supported and that all plumbing work conforms with the requirements of the local authorities. See that all joints are carefully made and are water-tight.

(27) Keep careful account of any extra work, in order to be able to explain satisfactorily, and check all extras. Keep a daily record of the progress of the work, including force employed, in a separate field book for each job. Note in same if any disputes arise, their settlement; also variations, if any, from the plans or specifications; at completion of the work, correct all plans and specifications to agree with work, as constructed, and make a final report showing costs, where required.

(28) A careful and complete record in detail must be kept of the cost of all alteration or repair work, wherever work of this nature occurs in any part of the construction. Also a detail of the cost to reproduce any and all structures that may be torn down, removed or otherwise destroyed shall be kept; and a report in full shall be sent immediately to the Engineer.

Water Works

Method of Making a General Investigation and Efficiency Test of the Lorain, Ohio, Water Works—Results and Recommended Improvements.

An investigation of the municipal water works at Lorain, Ohio, was recently made by Mr. Philip Burgess, Consulting Engineer, Columbus, O. The report submitted by Mr. Burgess contains a large amount of data of general interest and a considerable portion of it is here published. This matter should be of special interest to water works superintendents, managers, and others engaged in the operation or supervision of city water works. Reading between the lines, the desirability of making such investigations is clearly indicated. The scope of the investigation is outlined in the articles of agreement as follows:

SCOPE OF INVESTIGATION.

1. To determine the actual present total cost of obtaining, purifying, and delivering water to the consumers of the city of Lorain, including both the average total monthly cost and the average cost per 1,000,000 gals. delivered.

2. To determine the average proper cost of obtaining, purifying, and delivering water, which cost should properly be incurred were the water works properties operated in a manner such as to give as great an economy as it is reasonable to expect.

3. To determine what changes or modifications in equipment, apparatus, materials, fuel, labor, and all other such elements may be made to give greater economy or better efficiency of operation of the water works properties.

DESCRIPTION OF LORAIN WATER WORKS.

The municipal water system now in use at Lorain comprises an intake, water purification plant, boiler and pumping station, distribution system, and booster pumping station. The several parts, or elements, of the water works properties will be taken up in turn and will be discussed principally along the following lines: (1) Description of equipment provided; (2) operation of equipment; (3) efficiency of equipment; (4) cost of operation; (5) changes required to obtain higher efficiencies or decreased operating costs.

INTAKE.

Description.—During 1913 there was completed a new 48-in. cast iron intake pipe extending outside and to the west of the government break-water. This new intake was required because the former intake was so located that, during periods of easterly winds, the polluted waters of the Black River were carried by the lake currents over the intake pipe and, at such times, considerable difficulty was encountered in providing a water of proper sanitary quality. The new intake is so located that the present water supply is not affected materially by the lake currents. The old forebay has been abandoned and the new intake pipe is connected directly to the suctions of the low service pumps in the filter house.

Efficiency.—The new intake has not been in use for a length of time sufficient completely to demonstrate its worth but the recent operation of the plant proves that the raw water pumps are operating more smoothly and efficiently with the new than with the old intake; moreover, there is a marked improvement in the quality of the raw water pumped to the filter plant. Continued operation of this plant will doubtless show a greater economy of operation due to the use of less wash water and less chemicals required to treat the water. Moreover, in the future no difficulty should be incurred in discharging from the filter plant continually a water of good sanitary quality. In general the new intake has improved the quality of the water supply and has decreased the operating cost of the filter plant.

WATER PURIFICATION PLANT.

Description.—The water purification plant comprises two low service centrifugal pumps each of 6,000,000 gals. daily capacity, driven by steam turbines; two settling basins, the one holding 390,000 gals. and the other 190,000 gals.; six rapid sand filters with a combined area of 2,664 sq. ft. and a nominal capacity of 6,000,000 gals. daily; one steam turbine driven, centrifugal pump of 4,000 gals. per minute capacity provided to wash the filters; a clear well of 290,000 gals. capacity provided beneath the filters and pipe gallery; one turbine-driven centrifugal pump provided to remove the sludge and water from the settling basins; together with storage tanks, solution pumps, and controlling devices for applying the necessary chemicals.

Operation.—During the operation of the filter plant, raw water into which the chemical solution is introduced is pumped from the lake into the settling basins where it is retained theoretically for an average period of approximately 1 hours; from the basins, the treated water flows by gravity through the filters into the clear well from which it is pumped by the high service pumps into the distribution system.

Efficiency of Water Purification Plant.—The filter plant is in the immediate charge of Mr. Thomas L. Tristrum, city chemist, who makes daily tests of the quality of the raw and filtered waters. These tests are reported to the engineering department of the state board of health on blanks prepared in accordance with the recommendations of this board. The results of the daily tests show that during the month of November, 1913, the average number of bacteria in the raw water was 1,775 per c. c. and in the filtered water was 55 per c. c. The water delivered from the filter plant has been continually clear and of excellent quality from a sanitary standpoint. In general the results of all tests indicate that the filter plant is providing a very satisfactory effluent.

Cost of Operation.—During October, 1913, there were delivered to the consumers 108,600,000 gals. of filtered water. The cost of operation of the filter plant, not including the cost of pumping the water and the chemical solution, which will be discussed later, is shown in Table I.

The total cost of operating the filter plant during the month of November, exclusive of pumping, was $759, or $7.21 per 1,000,000 gals. delivered to the consumers.

This figure is higher than that generally incurred by the operation of purification plants similar to that provided at Lorain. A very considerable part of the cost of operation is the labor expense. The location of the filter plant, apart from the boiler and pumping station, is such as to require a distinct and separate labor organization. In other words, when the new filter plant was constructed in 1906 no attempt was made so to place the filter plant that its operating cost could be reduced to a minimum, but the filters were placed upon the coagulating or settling basins previously constructed by the city at such an elevation as to permit a gravity flow of water from the lake into the basins. Properly the new filter plant should have been placed at a higher elevation.

TABLE I.—COST OF OPERATION OF LORAIN FILTER PLANT DURING NOVEMBER, 1913.

(Exclusive of Pumping Expenses.)

Superintendent's salary	$120
Three filter operators at $70 each	210
Alum, 29,600 lbs. at 0.895 cts.	265
Chief engineer's salary, one-half, at $100	50
Supplies and repairs (from quarterly report)	114
Total cost of operation	$759

Moreover, an examination of the walls of the settling basins indicate that they were badly cracked and are leaking, a condition which is aggravated and made more serious by the regular practice of emptying the basins every four to six weeks for cleaning purposes. When emptied, of course, the dirt pressure on the outside of the basin walls is not balanced by the water pressure on the inside, so that emptying the basins places a sudden heavy load on the already damaged walls. Experience shows that it is necessary to empty and clean the basins every four to six weeks.

Recommendations of Changes of Construction and Operation.—While it is the opinion of the writer that the present operation of the plant is as efficient and economical as it is reasonable to expect under present conditions, there are several changes especially of construction which are believed to be very necessary and important.

Possibly the most important feature which requires immediate attention is the condition of the walls of the settling basins. This condition can be made of least effect by providing a sludge disposal system to remove the precipitated sludge from the basins without emptying them. Such a system would comprise lines of perforated soil pipe laid on the bottoms of the basins and carried up to a point about 5 ft. below high water in the basins. The discharge from these pipes should be controlled by quick-opening valves which would permit a sudden flushing out of the sludge every day using a small quantity of water. Such a sludge system probably could be provided at a cost of about $800. It would tend to prevent further damage to the walls of the basins and would, moreover, very materially improve the efficiency of the basins because the accumulations of sludge, which now at times are carried over onto the filters, would not be possible. Present practice requires that the basins be cleaned about every six weeks, but, with the sludge system in use, it would probably be necessary to empty the basins at intervals of not less than one year. Moreover, there would result a considerable saving of coal owing to the elimination of the fuel expense incurred by pumping the sludge and water from the basins as is now required when they are cleaned.

Examinations of the filters indicate that there has been a considerable loss of sand from the filters by washing. It is recommended that this sand be replaced using filtering material obtained from the filter tanks in the old house. A new sand delivered at Lorain in the filters would cost probably at least $4.50 per cubic yard, while the sand from the old filters should be placed in the new house at a cost not greater than $1.50 per cubic yard.

As previously stated, the filters are washed by a centrifugal steam-turbine driven pump. No duplicate apparatus is provided and, in case of a breakdown to this equipment, the filter plant would have to be by-passed and raw lake water furnished the city. It is, therefore, recommended that a wash water line be constructed from the clear well beneath the old filter house to the present wash water line in the new filter house. Such an arrangement would cost about $600 to install and would decrease the present cost of operation of the filter plant at least $350 annually on account of the elimination of the steam required to drive the wash water pump. It is apparent that the new wash water line would soon pay for itself and would, moreover, provide a duplicate wash water supply which is desirable in case of a breakdown to the present wash water pump.

There are two other important features of construction and operation of the filter plant which should be noted. These are the use of a germicide, such as hypochlorite of lime, to treat the water, and the use of deeper beds of coarse gravel in the filters. It is probably true that, with the use of 10 lbs. of hypochlorite of lime per 1,000,000 gals. of water filtered, the quantity af alum applied could be reduced to not more than 200 lbs. per 1,000,000 gals., thus reducing the cost of chemicals from $2.52 to $1.90 per 1,000,000 gals., and reducing the monthly cost of operation by $65. The only objection to the use of the germicide is the danger of possible occasional odors or tastes in the filtered water. During the past year, filter No. 6 has been overhauled and a deep layer of comparatively coarse gravel provided beneath the filter sand. Experience with this filter, and the writer's general experience at other plants, shows that such gravel layers permit more efficient operation of the filters and especially more economical washing. It is recommended that, in the future, such gravel be provided whenever it may be necessary to overhaul a filter.

Since the installation of the new filter plant in 1906, the old filter plant has been maintained ready for operation. This requires some expense to keep the old tanks tight and full of water. Moreover, it is difficult to see how the old tanks ever can be of any service to the city. It is, therefore, recommended that the old filter plant be disposed of, except the clear well, which may properly be used for the storage of filtered water required for washing the filters as recommended and, also, to carry over the peak loads on the filter plant during period of unusually large consumption, which at times occur especially in periods of continued hot dry weather. The filter sand also may be removed as required to the new filters and the rest stored for future needs.

BOILER PLANT

Description.—One of the most important elements of any power plant is the boiler equipment, and it is difficult to understand why the boiler plant at the main pumping station has been so neglected. The plant comprises two horizontal return-tubular boilers each 6 ft. diameter by 18 ft. long, with a nominal capacity of 150 H.P. and one two-furnace, horizontal, marine type boiler 10 ft. 6 ins. diameter by 11 ft. long with a nominal capacity of 300 H.P. The first two boilers were installed in 1894 and are therefore 19 years old, while boiler No. 3 was installed in 1907 and is 6 years old. The pressure allowed on boiler No. 1 is 90 lbs. per square inch, on No. 2 is 81 lbs. per square inch, and on the marine boiler No. 3 is 110 lbs. per square inch.

The piping arrangement for discharging steam from the boilers is very unsatisfactory because the discharge from boilers No. 1 and No. 2 is directly into a steam header, and it is impossible to obtain steam from these boilers except from the header. The header is built of steel pipe 14 ins. diameter, 51 ft. long, installed in 1895. The discharge from boiler No. 3 is either into the header or directly into the steam line to the filter house. Unfortunately, the steam header is so old and worn out that it is no longer safe to use it. Nevertheless, the arrangement of steam piping is such that it has to be used constantly and cannot be put out of service for renewals or repairs. It is important to note that, in case of a break to the steam header, the entire plant will be put out of service, yet in its present condition, the header may give out at any time. It also leaks badly and wastes considerable live steam. Boiler No. 2, which can carry but 81 lbs. per square inch, has been condemned and must be renewed during the coming year.

Efficiency of Boilers No. 1 and No. 2.—Owing to the present condition of boilers No. 1 and No. 2, which should have been removed some time ago and must be removed in the immediate future, no attempt was made during this investigation to run a detailed efficiency test on these boilers. From the past records of the plant data were obtained on the gas consumption and water pumped when these boilers were in service. Gas burners were placed under boilers No. 1 and No. 2, but not under No. 3, which was reserved for coal in case of an inadequate gas supply.

The average fuel cost of pumping with gas, using boilers No. 1 and No. 2, during 1912 was $5.90 per 1,000,000 gals., and during 1913 has been $5.32 per 1,000,000 gals. There is no explanation at hand for the considerable reduction of fuel cost during 1913 except that, as the chief engineer states, he has endeavored to go over the plant and to keep it in repair and especially to keep the steam traps in as good working order as is possible. These traps are of the bucket type and frequently are out of repair and permit a considerable waste of live steam. The increased economy of the plant may readily be attributed to the repairs of the traps which are worn out and should be replaced with efficient traps of better design. In general it may be said, however, that the efficiency of boilers Nos. 1 and 2 is less than should properly be obtained, because the allowable steam pressure is less than that which should be carried on the boilers to give economical operation.

Efficiency of Boiler No. 3.—The efficiency of boiler No. 3 has been very carefully investigated by duty or efficiency tests of about eight hours duration. The principal features of the results of the tests are shown also in Table II.

TABLE II.—RESULTS OF EFFICIENCY TESTS OF BOILER NO. 3.

	No. and date of test		
	1. Nov. 15.	2. Nov. 17.	3. Nov. 29.
Horsepower developed	46	177	183
Water apparently evaporated per lb. of coal under actual conditions, lbs.	4.51	7.70	7.48
Equivalent water actually evaporated from and at 212 degrees F. per lb. of coal as fired, lbs.	5.43	8.58	8.71
Equivalent evaporation from and at 212 degrees F. per lb. dry coal, lbs.	7.58	8.94	9.08
Equivalent evaporation from and at 212 degrees F. per lb. combustible, lbs.	6.55	10.41	10.66
Heat value per lb. of dry coal, B. T. U.	13,700	13,700	13,700
Efficiency of boiler, including grate, pct.	39.5	63.3	64.3

There are a number of interesting conclusions which may be drawn from Table II, as follows:

(1) That 9 lbs. of water from and at 212° F. are evaporated per 1 lb. of dry coal, under the usual operating conditions. This indicates a reasonable economy of operation especially in view of the fact that no attempt was made during the test to secure ideal conditions. Somewhat better results could be obtained were the two furnaces fed alternately instead of at the same time as was the practice during the tests. In other words, no attempt is usually made to obtain the best combustion which is possible with alternate firing of the two furnaces. Although alternate firing would require more constant attention on the part of the fireman, it is especially recom-

mended when the boiler is working under high loads.

(2) That 46 boiler horsepower is required during the day to operate the filter house, including all losses from the main steam line, and the steam required to operate the raw water pump and the chemical solution pump. The steam and coal consumption of the filter house, therefore, is 25.6 per cent of the total steam or coal required to run the entire pumping station.

(3) That, under the usual operating conditions which were maintained during the tests, the average boiler efficiency was 63.8 per cent. Although a somewhat higher efficiency might be obtained with alternate firing of the furnaces, the results are thought to be as good as those generally obtained from boilers of this type. During the tests, it was apparent that there was a poor circulation of water in the boiler, which was comparatively cool at the bottom. This is characteristic of marine boilers, which are not the best type for general use but are of advantage where space is limited.

(4) That, as would be expected, the efficiency of the boiler in generating steam is very much less with small than with large loads. This fact, of course, means that it is desirable to pump water at a constant rate, which is not possible at Lorain on account of the lack of a storage reservoir in the distribution system. Under present conditions at Lorain, the water consumption at night is about 75 per cent of the average daily consumption, and the water consumption during the day averages about 1.25 per cent of the average daily consumption. This means that the horsepower developed during the tests was 1.25 per cent of the average horsepower developed daily by the boiler. The average horsepower developed by the boiler, therefore, is 144, and the average daily efficiency of the boiler is somewhat less than 63.8 per cent on account of changes in the rate of evaporation.

(5) That in general, while the marine boiler is not the type best suited for the needs at the plant, the results accomplished by the boiler are as good as may reasonably be expected with the present equipment. Unquestionably better results would be obtained were it possible to operate the boiler at a higher pressure than is now possible with the present header in use.

Recommendations of Changes in Boiler Equipment.—It is recommended and is in fact absolutely necessary for the continued operation of the plant that steps be taken immediately to replace the steam header and the two return tubular boilers. These boilers are undoubtedly of the type best suited for the service at Lorain and could be replaced at a cost of about $11 per horsepower. Some difficulty will be encountered in keeping up the service during the replacement of the steam header, which will doubtless have to be replaced in sections and temporary steam lines provided to operate the reserve pumps during repairs to the header. Renewals of steam piping will probably cost about $500.

Too much importance cannot be placed upon the necessity of making such repairs at the earliest possible moment, because failure of the present header would mean closing down the water plant, and might also cause considerable damage at the plant. Incidentally, it should be mentioned that the past leakage of live steam from the header probably has cost more than the replacement of the piping several times over.

PUMPING EQUIPMENT.

Just as it has been shown that the boiler equipment at the pumping station has depreciated through age and use, and is in need of immediate repairs and renewals, so is it true that the pumping equipment is very inadequate for the needs of the city, both at the present time and in the immediate future. The present high service pumping equipment comprises one Gordon horizontal, compound, duplex pump of 2,000,000 gals. daily capacity, installed in 1894; one Holly Gaskill horizontal, duplex, compound, crank and flywheel, high duty pumping engine of 5,000,000 gals. daily capacity against a head of 95 lbs., installed in 1896; and one Holly, horizontal, duplex, compound, crank and flywheel pumping engine of 5,000,000 gals. daily capacity against a head of 20 lbs., installed in 1898. This pump was provided for low service and to lift the water from the lake into the old filter plant. It is surprising to note that the only high service pumping unit which is available for constant service is the big Holly Gaskill engine, which is used continuously, day after day, and is shut down for necessary repairs or adjustments, perhaps, 24 hours out of a year's run. It is fortunate for the city that this pump is unusually well built and durable and has been able to keep up the service demanded of it.

The other two units are not worth considering as emergency or duplicate equipment except at times of extreme necessity. The small Holly is an excellent pump for the service for which it was built, namely, 20 lbs. water pressure at a piston speed of 146 ft. per minute. The steam end is too small and the pump too lightly built for service under the water pressure usually carried in the distribution system. The difference between the large and small Holly pumps is best indicated by the fact that the steam end of the big Holly is designed for 180 H.P. and that of the small Holly for 40 H.P. The small Holly pump already has been badly damaged by using it on high service at times when the big Holly engine was out of service for necessary repairs.

The Gordon pump is of too small capacity to be of much value as an emergency or reserve equipment. Moreover, it is of obsolete design for the service required at Lorain. Such pumps when new give a duty of not more than 40,000,000 and the pump at Lorain in its present condition, would give a duty of not more than 15,000,000. This means that pumping with this engine requires several times as much coal as pumping the same quantity of water with the big Holly engine. Any further discussion of the value of this engine for services required at Lorain would be superfluous.

Test of Holly Gaskill Pumping Engine.—The efficiency of the Gaskill pumping engine was determined carefully by two duty tests of the plant and by indicator tests on the steam and water ends of the pump. The duty tests and indicator cards were taken just as the pump was found without any attempt to obtain ideal conditions. This pump was installed in 1896 to operate under the following conditions: steam pressure, 75 lbs.; average water pressure, 95 lbs. for domestic service and increased to 135 lbs. for fire protection; stroke, 3 ft.; piston speed, 100 ft. per minute; capacity, 5,000,000 gals. daily, with guaranteed duty of 105,000,000 ft. lbs. per 100 lbs. coal based on evaporation of 10 lbs. of water per pound of coal from a feed water temperature of 125° F.

It is apparent that the steam end required is considerably larger than is usually provided for pumps of such capacity. It is of interest to note that the pump has been overhauled but once in recent years, namely, in April, 1911, when new water plungers were installed. Numerous indicator cards were taken during periods of both large and small consumption of water and showed substantial agreement.

Before any determination of the work done by the pump could be made, it was, of course, necessary, first, to determine, as accurately as possible, the capacity of the pump per revolution, and the pressure against which the pump operates.

Capacity of Pump per Revolution.—No meter is available on the main discharge line from the pumping station to measure the quantity of water pumped into the distribution system, and it was, therefore, necessary to determine the pump capacity in some other manner. The method used was to measure the quantity of water passed through the filters into the clear well, whence it was pumped into the mains by the Holly Gaskill engine. The measurements of the quantity of water filtered were taken on three days of eleven hours each, at one half hourly intervals. The results obtained on the three days' test gave a substantial agreement of 235 gals. per revolution. The theoretical displacement of the pump is 235.7 gals. per revolution. It is difficult to explain why there was no more apparent slippage through the pump but the measurements of the quantity of water delivered are believed to be accurate and a displacement of 235 gals. per revolution was used in all computations.

It is of interest to note that this figure is considerably larger than that previously used by the engineers in calculating the water delivered by the pump, namely 216 gals. per revolution.

Water Pressure at Pump.—In order to determine the water pressure against which the engine operates, two accurately calibrated pressure gages were installed, the one at the air chamber of the pump, and the other in the boiler room connected to a 2-in. water line from which there was no draft during the test. Readings of these two gages were taken for 10 hours at intervals of 20 minutes; also of the old water gage on the board in the pump room end of the pressure recording gage also in the pump room. It was learned that the old gage on the board is correct within 1 lb. pressure at 60 lbs., while the recording gage is 11.3 lbs. high at 70 lbs. Unfortunately, it had previously been assumed by the engineers at the plant that the old gage on the board was incorrect and that the recording gage was all right. It has been customary to maintain 70 lbs. domestic pressure on the distribution system as shown by the recording gage so that, as a matter of fact, the pressure generally maintained is not 70 lbs. but slightly less than 59 lbs. This fact is of considerable importance, especially because, at times of high draft, which occur frequently in period of continued hot weather, the pressure in several parts of the distribution system has proven inadequate.

Indicator Tests of Gaskill Pumping Engine.—The results obtained by indicating the big Holly pumping engine are of considerable interest. In spite of the fact that the engine has not been overhauled for 2½ years, the results show that the valves in the steam and water end are in unusually good condition. The indicated horsepower of the steam end was 123.35, while the horsepower delivered as effective work by the water end was 104.7. The mechanical efficiency of the pump was, therefore, 84.9 per cent. This figure agrees well with the average efficiency which may be expected of pumps of this type. In general the results indicate that the valves in both the steam and water ends are in fair adjustment. It is apparent, however, that the left side of the pump is in a somewhat poorer condition than the right side and does less effective work. This may be accounted for in part by the arrangement of the discharge piping, which offers more resistance to the discharge from the left than from the right side of the pump.

Duty Tests of Holly Gaskill Pumping Engine.—As previously stated, no attempt was made during the duty tests to obtain ideal conditions, but the pump was tested when operated in the usual manner. It should be stated also that the results obtained are more accurate as station duties than as duty tests of the pump. The measurements of steam delivered to the pump were obtained by metering the water supplied to the boiler and correcting the total quantity of water evaporated for the amount of steam discharged from the drip on the steam header into the feed water heater; also for the steam passed through the engine driving the light machine and into the header; also for the steam delivered to the filter house, as had previously been determined. Owing to the numerous corrections necessary, the results obtained, while of considerable value as indicating the station duty with the Holly Gaskill engine in service, are perhaps not as accurate as could be obtained were it possible to weigh the condensate from the pumping engine.

The duty trial made on November 17 indicates that the pumping engine developed a duty of 50,300,000 ft. lbs. and the test on November 29 that it developed a duty of 42,500,000 ft. lbs. The average duty developed was, therefore, 46,000,000 ft. lbs., a figure which represents undoubtedly very nearly the average duty which is usually developed by the pumping engine.

While the duty developed was less than one-half of the guaranteed duty, it should be noted that the average water pressure was but 59 lbs. per square inch, and not 90 lbs. for which the pump was designed; also that the average piston speed was 72 ft. per minute, or 28 ft. less than the nominal speed. It is doubtless true that considerably higher duty would be obtained by the pump were it thoroughly overhauled and were it operated under more nearly ideal contract conditions.

Duty Tests of Raw Water Pumps and Turbines.—The steam consumed by the raw water, turbine-driven centrifugal pumps was determined by metering the feed water and hence the evaporation from boiler No. 3, which was placed in service to the filter house exclusively; by condensing and weighing the discharge from the steam traps on the main steam line, and by condensing and weighing the discharge from the chemical solution pumps.

It was learned that the weight of water discharged from the two traps was 25 lbs. per hour; also that the weight of steam consumed by the chemical solution pumps was 75 lbs. per hour. The total weight of steam consumed per month by these two services is, therefore, 72,000 lbs. at a fuel cost of $9.15 per month.

The balance of the steam delivered to the filter house was used to drive the raw water pump. The results indicate that the turbine and pump developed a combined duty of 11,700,000 ft. lbs., at a steam pressure of 71.6 lbs. The turbines are designed for a steam pressure of 90 lbs. The results obtained are very satisfactory in view of the operating conditions. The steam consumption would, however, be reduced at least one-half, if the turbines were operated condensing and under 90 lbs. steam pressure for which they were designed.

COST OF OPERATION OF BOILER PLANT AND MAIN PUMPING STATION.

The cost of operating the boiler plant and pumping station is composed of the following items: Labor, fuel, oil and supplies, repairs and maintenance.

Labor.—The station is operated on three shifts of eight hours each. The entire operation is in charge of a chief engineer, who has on his staff three assistant engineers, three regular firemen, one extra fireman, a machinist and two helpers. The total monthly labor cost, chargeable to the boiler and pumping station, is as follows:

Chief engineer (½ time)	$ 50
3 assistant engineers at $80	240
3 firemen at $65	195
2 machinists at $65	130
2 helpers at $60	120
Total monthly labor cost	$735

The total monthly cost of labor in the boiler plant and pump station is, therefore, $735 and the average cost of labor per 1,000,000 gals. pumped during 1913 has been $7.

Fuel Cost.—The cost of fuel, when gas was used, has been accurately recorded by readings taken daily of the gas meter. The practice has been to operate for 15 days each month using gas under boilers Nos. 1 and 2 and for 15 days using coal under boiler No. 3. The records indicate a considerable variation in gas consumed per million gallons pumped, and a marked reduction in the last months shown for 1913. There is also a noticeable increase in the cost of gas during the colder months of the year. The results indicate that the average cost of gas fuel per 1,000,000 gals. pumped in 1913 was $5.32.

Less accurate data are available to indicate the cost of fuel when pumping with coal. Records are kept of the number of wheelbarrow loads of coal used, but they are not weighed, and the average weight is assumed to be 300 lbs. Data obtained during the evaporation tests of the boilers indicate that during these tests 2.08 tons of coal were required to pump 1,000,000 gals. of water. It is probably reasonable to assume that the average coal consumption is very nearly 2.25 tons per 1,000,000 gals. pumped, and that the cost of fuel, therefore, averages $4.95 per 1,000,000 gals. pumped.

It is of interest to compare this cost with the past costs of coal for the years 1907, 1908 and 1910 before gas was used and when the annual coal consumption was computed from the total amount purchased. The average cost of coal per 1,000,000 gals. pumped was $4.23 during these years. As coal has subsequently increased in price, it is apparent that there is no material change in the quantity of coal consumed and that the estimated present cost of $4.95 per 1,000,000 gals. is reasonable.

The average monthly water consumption during the year 1913 has been 105,000,000 gals., so that the average present monthly cost of fuel is estimated as follows:

Monthly cost of fuel using gas—105x$5.32—$557
Monthly cost of fuel using coal—105x$4.95—$519

In general, present data show that it is cheaper to use gas for fuel in the summer months and coal in the winter months.

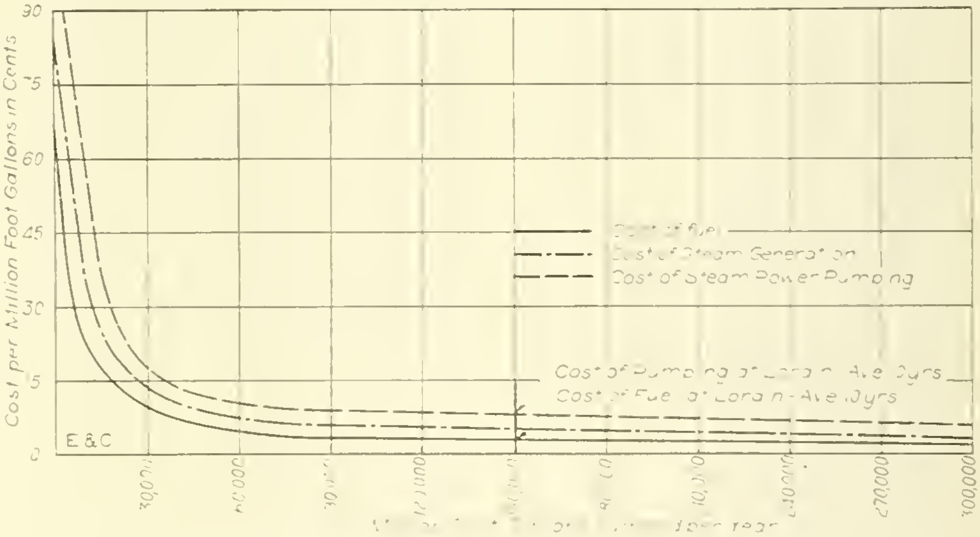


Fig. 1. Cost of Pumping by Steam Power in 35 Wisconsin Water Utilities, Showing Relation to 10-Year Average at Lorain, O.

Oil and Miscellaneous Supplies.—The cost of miscellaneous supplies, such as oil, boiler compound, waste and rags, electric current, tools, etc., necessarily varies considerably from time to time. The records indicate, however, that the average cost of such supplies per month is about $38, or 36 cts. per 1,000,000 gals. pumped.

Repairs and Maintenance.—The cost of repairs and maintenance required at the pumping station and boiler plant also is a variable quantity. The average cost of such repairs is estimated from the past records at $97 per month, or 92 cts. per 1,000,000 gals. pumped.

Recommendations of Changes in Pumping Equipment.—As previously stated, the conditions at the pumping station require immediate attention to betterment, because there is no adequate pumping unit provided to maintain service in the event that the Holly Gaskill pumping engine is out of service for either repairs or renewals. A breakdown of this unit would seriously cripple the pumping station and might cause an interruption or suspension of adequate service. The Holly pumping engine is giving very good service, especially in view of its age.

It is recommended that additional pumping units of adequate capacity to meet all demands that may be put upon them be installed at the earliest possible date. The [illegible] pumping engine should be removed and disposed of. This will give ample space in the present building for the new equipment which should comprise two centrifugal pumps, driven by steam turbines, the one of four and the other of six millions gallons daily capacity against a head of 80 lbs. water pressure. The pumps should be so connected up to the distribution system that they may be operated either in parallel or in series.

The turbines should be of the variable speed type, thus giving the required range of operating conditions which now maintain at Lorain. They should also operate condensing, at 90 lbs. steam pressure, under which conditions a combined duty of at least 65,000,000 ft. lbs. should be obtained.

With such equipment it will be possible to deliver water under 80 lbs. pressure into the distribution system at a cost for fuel, but little if any greater than that now required to deliver it under the present pressure of 59 lbs. The cost of installing such equipment complete should be not more than $5,500.

TABLE III.—PERIODS OF MAXIMUM CONSUMPTION OF WATER AT LORAIN, OHIO.

Date, 1912	Daily consumption of water, gals.	Rate of m'xim'm consumption for 4 hrs. daily.
June 30	4,331,000	6,500,000
July 1	3,854,000	6,000,000
July 2	4,129,000	6,500,000
July 3	4,261,000	6,500,000

The necessity for such new pumping equipment is apparent, not only because the present Holly Gaskill pumping engine cannot properly be operated continuously without renewals and repairs, but also because at times of continued dry hot weather it is impossible to maintain the required supply with this engine. The extent of such periods is well indicated in Table III, which shows the dates, daily consumption of water and maximum load on the Holly pump for periods of about four hours daily. It will be recalled that this pump has a nominal capacity of but 5,000,000 gals. daily.

During these periods of maximum demand, the Holly pump was operated at the maximum possible speed, notwithstanding which fact there were numerous complaints from several parts of the distribution system on account of inadequate pressure. In case of a fire at such times, the city would be without adequate protection. Such conditions have arisen in the past and will again in the future unless the pumping equipment is increased.

BOOSTER PUMPING STATION.

Owing to the lack of adequate water pressure in South Lorain a booster pumping station was constructed in 1910 about 2.6 miles from the main pumping station. This station contains two 8-in. centrifugal pumps, each driven by a 10 H.P. tandem compound gas engine. It is the present practice to operate one of these units daily, from about 5 a. m. to 10 p. m.

No means are available to measure the quantity of water pumped by this station. The quantity of gas consumed is indicated by a meter which is read daily. The annual cost for gas at the booster station is shown to be $592.26. Numerous readings of the gages on the pump indicate that the pressure in the 12-in. main pipe line is increased from about 20 lbs. to 53 lbs. per square inch.

The meter readings indicate that the one engine consumes about 300 ft. of gas daily more than the other, a fact which is readily accounted for by the present condition of engine which should be overhauled and repaired.

Rate of Gas Consumption.—In order to measure the amount of work done by the engines one of the units was tested by means of a rope brake. The tests indicate that the engine tested will develop 40 H.P. with difficulty. The gas consumption per horsepower is as follows:

When operating at a rate of 20 H.P. gas consumed = 14.7 cu. ft. per H.P. hour.

When operating at a rate of 30 H.P. gas consumed = 11.6 cu. ft. per H.P. hour.

The engine tested was in poor repair owing to leaks, but it is apparent that the engines are capable of doing very economical work.

Cost of Operating Booster Pumping Station.—Following is shown the annual operating cost of the booster station:

Labor, 3 men, $960 (after Jan. 1, 1914)....	$2,880
Fuel, gas	592
Miscellaneous supplies, repairs, etc........	100
Total annual operating cost..............	$3,572

The total cost of constructing the booster station was approximately $6,000, thus giving an additional annual cost of interest and depreciation (estimated at 7 per cent) of $420. The total annual cost of operating and maintaining the booster station is, therefore, $3,992, or 6 per cent of $66,500.

Recommendations.—In considering the work done by and the efficiency of the booster pumping station it should be noted that the extreme difference in elevation between the highest and lowest parts of the city of Lorain is not more than 85 ft. While it is true that, under the usual operating conditions, there is a loss of about 40 lbs. pressure between the main pumping station and the booster pumping station, the difference in elevation between the two stations is but 19 ft., equivalent to 21 lbs. pressure. It is apparent, therefore, that there is a loss of about 19 lbs. pressure between the two stations on account of friction losses in the pipe system. The problem of obtaining an increased pressure at South Lorain, therefore, properly requires a consideration of the possibility of increasing the capacity of the pipe lines, with a view of obtaining less friction losses. During the present investigation this matter was investigated thoroughly, as will be discussed later.

DISTRIBUTION SYSTEM.

In order to determine the adequacy of the distribution system, readings of the static and butt pressures were taken at several hydrants located at representative points. A table was prepared to show the locations of the hydrants tested, the elevations of the ground at the hydrants, the static pressure and the equivalent height of the free water surface at the hydrants, the differences in elevations between the hydrants and the pump at the main pumping station and, finally, the friction losses in the pipe system between the hydrants and the pumping station.

In general the results indicate that the distribution system is unbalanced and that much the heaviest consumption of water is on the line locally called the east 12-in. line. The present friction losses in this line are high and could be very much reduced by utilizing to better advantage the west 12-in. pipe line, which is practically parallel to the east pipe line and which at present is but little used.

Recommendations. In order to obtain more equal pressures in this distribution system it is recommended that the booster pumping station be abandoned and that additional pipe lines be constructed. All pipe required should be at least 12 ins. in diameter. The total length of 12-in. pipe line required is approximately 7,800 ft. and could be installed at a cost of not more than $12,000.

In order to maintain as efficient service at South Lorain as is now maintained by the booster station, in addition to the construction of the pipe lines, it will be necessary to increase the water pressure at the main pumping station about 20 lbs. per square inch or from the present pressure of 59 lbs. to 80 lbs. per square inch.

The operation of the booster station entails a present annual cost of $3,912. Under 80 lbs. pressure the Holly Gaskill pumping engine would operate at a higher duty so that, although the pump would do more work, the fuel cost would not be materially increased; also there would be no noticeable increase of other operating expenses at the main pumping station. The changes of operation and the addition of the new pipe lines would, therefore, save the city of Lorain annually at least $3,000 operating expenses. Moreover, there would result better water service throughout the entire city, including South Lorain, a fact which would be especially beneficial during fires and during periods of continued hot weather.

Before concluding the discussion of the booster station it may be said that, generally, the installation of a booster pumping station is required or warranted only where there exists such considerable differences of elevation between different sections of a city that it is not practical to operate the high service pumps at the pressure required to give adequate service to all parts of the city; or where the length of pipe lines is such that it is too expensive to provide pipes of adequate size sufficient to reduce the friction losses. Neither of these conditions exists at Lorain. In fact, the evidence clearly shows that the city could properly afford to spend over $60,000 in pipe lines before the construction of the booster station would be warranted. As a matter of fact but $12,000 is required to construct pipe lines which will give better water service to the entire city, including South Lorain, than is now furnished even with the booster station in operation. Consequently it is recommended that the booster station be abandoned, that the new pipe lines be constructed and that the domestic pressure maintained at the main pumping station be increased from 59 to 80 lbs. per square inch. It is difficult to understand how the booster station ever was constructed, because it eliminates only a part of the trouble experienced at Lorain on account of low water pressures and is neither the most satisfactory nor the most economical method of increasing the water pressure at South Lorain.

During the tests at the main pumping station it was noted that occasionally there was a very heavy draft from the pipe system, causing the pump to speed up rapidly and also causing a temporary reduction in water pressure of 10 to 15 lbs. This was said to be due to the fact that in the B. & O. R. R. yards engine tanks are filled directly from the mains. This practice should not be permitted and the railroad company should be compelled to provide a suitable elevated storage tank from which the engine tanks may be filled.

COMPARISON OF OPERATING COSTS AT LORAIN WITH COSTS AT OTHER WATER WORKS PROPERTIES.

A thorough examination of the past reports of the water works department at Lorain has been made with a view of determining the average past costs of operation. Unfortunately such records have not been kept, especially in recent years, in such complete form as is desirable. For comparative purposes, costs of operation per million gallons pumped and per capita at other water works properties are shown in Table IV. The costs shown include true operating costs exclusive of extraordinary expenses such as are incurred by extensions or replacements.

The cost of operation per million gallons pumped at Lorain has increased in recent years, and especially after the installation of the new filter plant in 1907 and of the booster pumping station in 1911. The average total cost of operation per 1,000,000 gals. pumped during the past 10 years has been $23.72 and the average per capita cost has been $1.17. During the last four years these costs have increased to $29 per 1,000,000 gals. pumped and to $1.20 per capita. The increase in cost may be attributed to the increased cost of fuel; to the gradual depreciation of the boiler and pumping plant, thus requiring increased labor and repairs for maintenance; to the increased quantity of chemicals required to treat the raw water which has become more polluted with the growth of the city, and to the increased quantity of coal required to operate the new filter plant as compared with the old and to operate the entire plant on account of the low steam pressure now carried on the boilers.

As previously discussed, the cost of operation of the water works properties may be materially reduced by installing new boilers and thus increasing the steam pressure and decreasing the cost of fuel and by eliminating the booster station.

A comparison of the operating costs at Lorain with those incurred at other water works properties indicates that the average past cost of operating at Lorain compares reasonably well with those of other similar plants. This is especially true of the fuel and pumping station expenses, as is shown in Fig. 1, indicating the costs of fuel and pumping at 35 water works properties in Wisconsin. As shown in Fig. 1, both the average fuel and the average pumping station expenses at Lorain during the past 10 years have been less than the averages at the 35 plants which have been investigated by the Wisconsin R. R. Commission.

TABLE IV.—ANNUAL OPERATING COSTS OF 16 WATER WORKS PROPERTIES.

			Annual operating cost.	
City—	Dates.	Total population.	Per 1,000,000 gals.	Per capita.
Ashland, Wis...	1904-09	12,150	$36.23	$1.25
Manitowoc, Wis.	1907	13,400	47.76	1.32
Janesville, Wis..	1909	13,800	49.96	1.30
Beloit, . Wis.....	1908	14,100	23.35	1.02
Chillicothe, O...	1908-12	14,500	.....	1.00
Marinette, Wis..	1909-10	14,650	26.17	1.09
Private water co. in Arkansas ..	1908-12	15,400	.....	1.57
Elyria, O.........	1910-12	15,500	34.88	1.47
Appleton, Wis...	1909	16,700	36.73	1.29
Fond duLac, Ws.	1907	17,800	20.31	0.77
Eau Claire, Wis.	1907	18,650	18.24	0.71
Private water co. in Western Pa.	1905-10	22,570	40.45	1.17
Green Bay, Wis.	1907	24,000	48.56	0.90
Battle Cr., Mich.	1908-12	25,270	26.20	0.72
Madison, Wis...	1908-12	25,460	49.70	1.28
Sheboygan, Wis.	1908	25,500	24.74	0.77
Lorain, O.	1900-10	22,070	23.72	1.17
Aver. of 16 cities above		18,090	34.51	1.10

RATES.

In conclusion it should be said that the writer has examined the reports of the water works department and also the report recently submitted by Ernst and Erns, accountants, Cleveland, especially to determine the sources of revenue from sale of water at Lorain. From these reports it is apparent that the present rates are poorly balanced. Before equitable rates for water service may properly be established it is necessary to determine all capacity and output expenses, including operation and overhead or investment expenses for all classes of service rendered, including domestic, industrial and municipal services. The industrial consumption of water at Lorain is 22.2 per cent of all the water pumped. It is, of course, a much larger percentage of the water actually sold. It is, therefore, very apparent that, before the water works department at Lorain can be put upon a sound financial basis, the rates to be charged for all services, including municipal service, should be accurately determined and enforced.

A New Sea Wall in Panama.—The Panama Railroad is now engaged in building a concrete sea wall along the front of the fill between the fire station at Cristobal and the end of the mole for the new piers to be used by the buildings of the steamship companies. The total length of the wall will be about 350 ft. It is composed of concrete cubes, one yard on a side, which are laid in brick style and faced above the water line with a wall of concrete one foot thick.

Lawrence Experiment Station Studies In the Purification of Water.

An article giving the results of certain water purification studies made at the Lawrence experiment station of the Massachusetts State Board of Health during 1911, was published in Engineering and Contracting of Jan. 8, 1913. The present article continues a report of the studies made during the year 1912, under the direction of H. W. Clark and Stephen De M. Gage, and is based upon information published in the recently issued annual report of the Board for 1912. The 1912 experiments on water purification were made on the slow sand filters of the city of Lawrence and on small experimental filters at the station. Studies were made of the effect on sand filter efficiency of changing the rates of operation, of the value of clarification by upward filtration through coarse material followed by sand filtration, and on rapid filtration with ferric sulphate as a coagulant.

LAWRENCE CITY FILTERS.

The source of the water supply of the city of Lawrence is the Merrimack River. Two filters are in use to purify this water. The older filter was constructed in 1893, and dividing walls separating it into three sections were built in 1902. The average depth of sand in this filter is about 4 ft., and the net filtering area, after deducting division walls, gate-chambers and lateral carriers, is about 2.2 acres. As originally constructed, this filter contained two different grades of sand, the portions of the filter immediately over the under drains being of finer sand than the remainder of the filter. Through the operations of scraping, washing and replacing sand, the two grades of sand have become quite thoroughly mixed in the upper layers of the filter, and at the present time this upper sand has an effective size of approximately 0.25 mm. This filter is not covered and has an earth bottom through which some ground water finds its way into the underdrains and becomes mixed with the filtered water. The average rate of operation during the past few years has been about 1,000,000 gals. per acre daily.

During 1906 and 1907 an additional filter was constructed to supplement the supply of filtered water from the old filter. This filter, which is three-quarters of an acre in area and contains about 4½ ft. in depth of sand of an effective size of 0.25 mm., is of concrete construction, has a concrete bottom and is covered. It was first put into operation on Nov. 4, 1907, but the filtered water was not used until Jan. 4, 1908. The average rate of operation during 1912 has been about 3,000,000 gals. per acre daily. The effluents from both of these filters flow into the same pump-well from which they are pumped into the distributing reservoir.

During the greater part of the year it is estimated that approximately half of the entire volume of filtered water which is pumped to the reservoir is supplied by the covered filter. As neither of the two filters is equipped with rate-controlling apparatus nor with loss of head gages, it is not possible to determine the actual rate of either filter at any time. During the winter months the ice accumulates upon the uncovered filter, and it is not always possible to scrape the surface of this filter when it becomes clogged; there is little doubt that at such times the quantity of water passed by this filter is greatly reduced and that the deficiency is supplied by forcing the covered filter. This is shown by the difference in the scraping record of the two filters during the winter. Between Jan 1 and March 15, 1912, the surface of the covered filter was scraped five times, or at an average interval of about once in two weeks. During the same time 14 of the 25 beds into which the uncovered filter is divided were scraped twice, 3 other beds were scraped once, and 8 beds, or about one third of the surface, received no surface treatment of any kind whatever.

The average number of bacteria in the effluent from the uncovered filter during the entire year was 36 per c. c., and in the effluent from the covered filter was 58 per c. c., these figures corresponding to removals of 99.6 and 99.3 per cent., respectively. An inspection of the daily results, however, shows a considerable difference in the bacterial quality of the effluents from these two filters during the winter months when the covered filter was being forced to make up the deficiency in the volume of water which could be passed through the older filter. Samples containing more than 100 bacteria per c. c. were occasionally obtained from the uncovered filter throughout the year, but the proportion of such samples from this filter was no greater during the winter than during the summer, and in none of these samples were the numbers of bacteria excessively high. In the case of the uncovered filter, however, while samples containing more than 100 bacteria per c. c. were found only occasionally during the summer, during the period from January 1 to March 15 the counts on more than one-half of the samples were above this limit, while during January only one sample was obtained which contained less than 100 bacteria per c. c., and the average for the month was about 200 per c. c.

EXPERIMENTAL SLOW SAND FILTERS.

Filters Nos. 8A and 343.—Filter No. 8A, 1/200 of an acre in area and first put into operation on Sept. 26, 1893, contained about 26 ins. in depth of sand of an effective size of 0.28 mm. at the beginning of the year. The average rate of operation during 1912 was 2,915,000 gals. per acre daily. During the year this filter was scraped to relieve clogging ten times, the total amount of sand removed by scraping being about 4.5 ins., an average removal of about 0.45 in. of sand at each scraping, or about ¾ cu. yd. for each 1,000,000 gals. of water filtered during the year. The average volume of water filtered between scrapings was about 80,000,000 gals. per acre, the average time between scrapings being about 29 days. The longest run of the filter was from February 28 to June 15, when the filter was operated at the prescribed rate for 75 days without scraping, about 218,000,000 gals. of water per acre being filtered in this period. The shortest run of the filter immediately followed the longest run, the interval before scraping was again necessary, being 15 days, during which about 37,000,000 gals. per acre were filtered.

Filter No. 343, 1/20000 of an acre in area, was first put into operation on March 27, 1908, and contained about 33 ins. in depth of sand of an effective size of 0.35 mm. at the beginning of the year. The average rate of operation during 1912 was 5,204,000 gals. per acre daily. The surface of this filter was washed 12 times during the year, the average interval between washings being 29 days and the average amount of water filtered between washings being about 117,000,000 gals. per acre. The longest period during which the filter could be operated at the prescribed rate was from April 27 to July 2, a period of 65 days, during which about 343,000,000 gals. of water per acre were passed by the filter. As was the case with Filter No. 8A the shortest run of the filter immediately followed the longest run, when during a working period of nine days about 48,000,000 gals per acre were filtered before surface washing was again necessary.

In point of bacterial efficiency Filter No. 343, constructed of coarser sand and operated at a higher rate, was superior to the older filter. As has been the practice at the experiment station, the sand removed from Filter No. 8A by scraping has not been replaced by clean sand according to the usual custom with large filters, and the inferior bacterial efficiency of this filter is undoubtedly due to the fact that the depth of the filtering medium has been reduced below the limit of safety. This effect is particularly noticeable as the result of scraping during the winter season, high bacterial counts being invariably obtained on samples collected during the first few days after the scrapings in December, January, February and March, although found less frequently following scraping during the warmer months. The necessity of maintaining a sufficient depth of filtering material—especially during the cold weather when the biological activity, upon which filters of this type depend for their efficiency, is at its lowest ebb—has been emphasized in these reports many times, and a striking example of the failure of an inadequate depth of material is furnished by the experience with this filter.

COMPARISON OF SAND FILTERS OPERATED AT DIFFERENT RATES.

Filters Nos. 417, 418, 419 and 420. On Dec. 28, 1910, four filters were put into operation in order to study comparatively the effect of the rate of operation upon the hygienic efficiency and operating economy of water filters of the so-called "slow sand" type. As originally constructed each of these filters contained 54 ins. in depth of sand of an effective size of 0.25 mm. Through the operations of surface washing and occasional scraping the depth of sand in these filters was somewhat reduced, and at the beginning of the present year the depth of sand in Filter No. 417 was 51 ins., in Filter No. 418, 49 ins. in Filter No. 419, 46 ins. and in Filter No. 420, 43 ins.

In the practical operation of water filters the question of the rate of filtration in its relation to the time and labor required to maintain the filter at the prescribed rate is of the utmost importance. A satisfactory purification of the water being the paramount issue, the filter by which that purification can be accomplished with the least care and expense is the one which is of the most practical value. From this view point the records of the operation of these four filters furnish a valuable and instructive lesson. As has been the custom with many of the experimental filters operated at the station during the recent years, it has been the practice to remove clogging matters from the surface of these filters when necessary by what is generally known as the "Brooklyn method"; that is, by washing the surface of the sand in place, the clogging materials being removed by a shallow current of unfiltered water flowing across the surface of the filter while the sand is agitated with a rake to a depth of about 1 in. During the year Filter No. 417, operating at a rate of 2,500,000 gals. per acre, was washed six times; Filter No. 418, operating at a rate of 5,000,000 gals., was washed nine times; Filter No. 419, operating at a 10,000,000 gal. rate, was washed 56 times; and Filter No. 420, which it was attempted to operate at a 20,000,000 gal. rate, was washed 108 times.

The average period during which Filter No. 417 would be operated without washing was about 65 days, and there were two intervals, April 3 to June 29, and August 19 to November 22, during which the filter was operated for periods of 81 days without surface treatment being necessary. The average volume of water filtered between washings was about 159,000,000 gals. per acre, while during each of the two 81-day intervals over 200,000,000 gals of water per acre were passed by the filter. The smallest interval between washings was 37 days, from December 16 to January 23, in which about 93,000,000 gals. per acre were filtered by this filter. The average interval between washings on Filter No. 418 was 37 days, and the average volume of water filtered between washings was about 190,000,000 gals. per acre. The shortest run of this filter was seven days, from February 21 to March 3, when only about 37,000,000 gals. of water per acre were filtered, while the longest run of the filter, lasting from March 3 to June 28, 111 working days, in which about 585,000,000 gals of water per acre were filtered, followed immediately after the shortest run.

On Filter No. 419 the average interval between washings was about five days, and the average volume of water filtered per washing was about 57,000,000 gals per acre. The longest run of this filter was from August 30 to October 22, a period of 43 days, during which about 397,000,000 gals. of water were filtered per acre. There were eight runs of this filter, however, which lasted only about 24 hours, and eight other runs when it was necessary to wash the filter after an interval of only two days. In fact, during the period from May 15 to June 26 it was necessary to wash this filter at an average interval of about once in 36 hours, and during this period the average volume of water filtered per run was only about 12,000,000 gals. per acre. It was necessary to wash the surface of Filter No. 420 on an average of about once in two days throughout the year. On five different dates it was necessary to wash this filter twice in one day, and 37 other runs of the filter lasted only about 24 hours. The longest run of this filter at any time during the year was only eleven days, and on only five runs during the year was the interval between washings five days or more. The average volume of water passed by this filter between washings was only about 51,000,000 gals. per acre, the maximum amount filtered during any single run being about 204,000,000 gals. per acre.

Comparing the average volumes of water passed by each of these filters between washings, it will be noted that the volume passed by Filter No. 418, which was operated at a rate of 5,000,000 gals. per acre daily, was about 20 per cent larger than that from Filter No. 417 which was operated at a rate only one-half as great, and almost four times as great as the average amounts per run filtered by Nos. 419 and 420 which were operated at rates two and four times as high, respectively. Both in the time interval and in volume of water filtered between washings a striking difference is to be noted between the two filters operated at the 5,000,000 and 10,000,000 gal. rates. It is quite evident, therefore, that somewhere between 5,000,000 and 10,000,000 gals. per acre daily lies the rate at which it would be most economical to operate filters of the type under discussion. With frequent attention to the surface it is possible to operate such a filter, as was shown by Filter No. 419, at a rate of 10,000,000 gals. per acre daily, although at somewhat increased expenditure for cleaning. Even with the very frequent surface treatment which it received, however, it was impossible to maintain a rate of 20,000,000 gals. per acre daily with Filter No. 420, and the average amount of water which could be obtained through this filter was less than 17,000,000 gals. per acre daily.

As was explained in the last report, the comparative bacterial results on these four filters have been somewhat disappointing, owing to the fact that secondary growths of bacteria have occurred from time to time within the filters, resulting in high numbers of bacteria in the effluents which have entirely obscured the relative removal of the usual water bacteria. For this reason the average room temperature counts on all of these filters are higher than those usually found in the effluents from properly designed and operated water filters, and the highest result is to be observed in the effluent from the filter which was operated at the lowest rate. In the elimination of the types of bacteria growing at body temperature, however, the best result was obtained with the filter operating at the lowest rate and the poorest result with that operating at the highest rate, although the differences are relatively small. While the average body temperature counts are not higher than those frequently found in many reasonably good water supplies, much higher counts were found at times, and in this respect the effluent from Filters Nos. 419, and 420 was at times far above the limit which is generally considered safe. In the removal of B. coli, a regular graduation is to be observed in the results, the relative occurrence of that organism in the filtered water increasing with the rates at which the different filters were operated. A similar effect is to be noted in the chemical quality of the effluents from these filters, in general the amounts of color, free ammonia and albuminoid ammonia being higher as the rate of filtration increased. Incidentally, also, the amounts of nitrates in these effluents also increase with the rates, although the reverse might have been predicted from the other chemical results.

DOUBLE FILTRATION.

Clarification by Upward Filtration through Coarse Material followed by Sand Filtration. Filters Nos. 380 and 286.—Filter No. 389, 1/2000 of an acre in area, was first put into operation on April 11, 1910. This filter is constructed of 6 ins. in depth of wood charcoal supported upon a ¼-in. mesh galvanized wire screen placed 12 ins. above the bottom of the tank. Overlying the charcoal and separated from it by a ⅛-in. mesh copper screen is 24 ins. in depth of broken stone pebbles of an effective size of 4.4 mms. The canal water enters at the bottom, passes upward through the filter material, and overflows through an orifice placed 3 ins. above the surface of the stone, the available loss of head with the filter in operation being about 10 ins. From April 6 to Dec. 1, 1911, this filter was not operated, but on December 2 it was again put into operation at a rate of 5,000,000 gals. per acre daily. On January 15 the rate was increased to 7,500,000 gals., and on February 1 the rate was again increased to 10,000,000 gals. per acre daily, at which rate the filter was operated during the remainder of the year. When the filter becomes clogged to such an extent that the water will not pass through at the prescribed rate, it has been the practice to remove the clogging matters by reversing the flow through the filter for a short time. Upon starting, the filter was operated for a period of 79 days before washing by reversed flow became necessary. Since that time, however, it has been necessary to reverse the current about once in six days, or 33 times during the year, the maximum period during which the filter could be operated without washing being about 18 days. The average volume of water filtered between washings has been about 65,000,000 gals. per acre, and the maximum quantity filtered during any single run was about 159,000,000 gals. per acre.

Secondary Filter No. 286 was first put into operation on Jan. 20, 1906. This filter is 1/20000 of an acre in area, and in 1912 contained about 31 ins. in depth of sand of an effective size of about 0.21mm. During the past few years Filter No. 286 has been operated as a secondard filter, receiving the effluent from pre-Filter No. 331. Beginning January 22, and continuing throughout the year, the effluent from upward-flow Filter No. 389 has been applied to Filter No. 286 at a rate of 5,000,000 gals. per acre daily. As was the custom with the earlier water filters at the station, the surface of this filter has always been scraped when necessary to relieve clogging, but since the filter has usually received water which has already been filtered, surface treatment has been required only at comparatively infrequent intervals. During the period from December 1 to July 2 the surface of this filter was scraped six times, or on an average of about once in 31 days, the average volume of water passed between scrapings being about 143,000,000 gals. per acre. After scraping on July 2 no further surface treatment of this filter was needed during the remainder of the year, a period of 147 working days, in which the filter passed over 600,000,000 gals. of water per acre.

The bacterial results obtained with Filter No. 389 after its long rest were much the same as would have been obtained with a new filter. During December and January, however, this filter gradually came into maturity, and during the remainder of the year the results were remarkable when the depth and character of the filtering material are taken into consideration. During the last ten months of the year the average reduction in the room temperature counts was over 92 per cent, the reduction in the body temperature counts was about 83 per cent, and about 45 per cent of the samples were free from B. coli, as shown by tests of 1 c. c. In March, April and June the room temperature counts on samples from this filter were as low as those which have been obtained on the effluents from many sand filters operated at much lower rates, while during May B. coli were not found in any of the samples. The body temperature counts, however, have never been sufficiently low to permit the effluent from this filter to be classed with those from good sand filters.

The effluent from Filter No. 389 has been collected and pumped at intervals to a supply tank, from which it has been applied to secondary Filter No. 286. After storage in these tanks an increase has been frequently noted in the room temperature counts, and also at times in the body temperature counts, and in the relative occurrence of B. coli in the water.

During the last ten months of the year the increase in the room temperature counts ranged from 75 to over 4,000 per cent, and averaged about 650 per cent. During five of these ten months there was a decrease varying from 7 to over 60 per cent in the body temperature counts on this water after storage, but during the remaining five months there was an increase in these counts varying from 8 to about 270 per cent. During July and October also, B. coli were found somewhat more frequently in the stored water than in the effluent from the filter. The multiplication of bacteria capable of growing at body temperature in this storage tank is of particular significance. It is quite generally recognized that there may be a large increase in the numbers of bacteria in water during storage, but it has generally been assumed that such bacteria were harmless saphrophytic forms, and that the types of bacteria which grow at body temperature were incapable of multiplication in water at the usual temperatures. This assumption has hitherto been based upon the results of qualitative tests for organisms of the colon type, and has been unsupported by regular quantitative counts. The results obtained in this experiment, however, are conclusive evidence that the types of bacteria which are determined on the body temperature plates can, and under certain conditions do, multiply in water during storage, and, furthermore, this evidence is supported by the results of the qualitative B. coli tests. As to whether any of the disease-producing types would multiply under the same conditions cannot be determined, but it is more reasonable to assume that they might do so than to assume that they are incapable of doing so. During the four months March to June, inclusive, the effluent from Filter No. 286 contained low numbers of bacteria and was generally of good quality. During the remainder of the year, however, the bacterial efficiency of the filter has been low, and high counts at both temperatures have been the rule. This may be attributed in part to the fact that there was practically no suspended matter in the applied water to build up a surface film on the filter, and that the applied water contained relatively large numbers of bacteria which were able to pass through the filter practically unchecked. This period of low bacterial efficiency coincides with the long period previously mentioned, when the filter was operated without surface treatment being necessary. The high numbers of bacteria in the effluent from the filter cannot be ascribed to growths within the filter as in the case of Filters Nos. 417 to 420, inclusive, previously noted, for although an increase in the bacterial content of the water during its passage through this filter was noted in a few instances, the specific types of bacteria always present when such growths have been noted in other filters were absent from the plates made from samples obtained from this filter.

RAPID FILTRATION WITH FERRIC SULPHATE AS A COAGULANT.

Filter No. 336.—Filter No. 336, 1/40000 of an acre in area and constructed of 24 ins. in depth of sand of an effective size of 0.35 mm., was first put into operation on Dec. 3, 1907. In previous years the water applied to this filter has been treated with sulphate of alumina and soda ash before filtration. In 1909 some studies with another filter indicated that a successful coagulation of the water could be obtained with ferric sulphate, and that with this coagulant the addition of soda ash or other alkali to the water was not necessary. Throughout 1912 ferric sulphate has been used as a coagulant for the water applied to Filter No. 336, the amounts added being varied to determine the effect of varying conditions in the process.

From December 13 to January 14, inclusive, the filter was operated at a rate of 100,000,000 gals. per acre daily, and ferric sulphate was added to the water in amounts varying from 1.5 to 2.25 grains per gallon. Operating at this rate there was a period of about 3½ hours for coagulation and sedimentation before the treated water entered the filter. With the smaller amounts of coagulant the removal of bacteria and organic matter by the filter was comparatively slight, while with the larger amounts the coagulant was only partly decomposed and the filtered water was badly discolored with iron. As it was evident that the time allowed for the reaction of the coagulant with the water was altogether too short, on January 15 the rate of the filter was reduced to 50,000,000 gals. per acre daily, and the coagulation period thereby increased to about 6½ hours. From January 15 to March 3 ferric sulphate was added to the applied water in the proportion of 4.5 grains per gallon; from March 4 to May 23 in the proportion of 3.5 grains per gallon; and from May 24 to July 24 in the proportion of 2.5 grains per gallon. During each of these periods the filter effluent was low in color, bright and of good appearance, and it was evident that the reactions between the ferric sulphate and the water were being completed in the coagulation basin. In all three periods, also, the removal of the types of bacteria determined at body temperature was entirely satisfactory, and B. coli were seldom found in the filtered water. The reduction in the room temperature counts, however, was not entirely satisfactory during the first period, although during the last two periods when smaller amounts of chemical were used the total bacterial efficiency of the process was over 99 per cent, and the effluent was uniformly of good bacterial quality. From July 24 to October 1 the amount of coagulant added was reduced to 1.5 grains per gallon. With the smaller amounts of chemical an increase occurred in the bacteria in both the effluent from the coagulation basin and the effluent from the filter, and most of the samples were of poor quality. During this period, also, the effluent from the filter was stained with iron, and it was apparent that the decomposition and removal of the ferric sulphate was incomplete. On October 2 the proportion of ferric sulphate was increased to 2.0 grains per gallon, and this was the amount used until the end of the year. During this period the removal of bacteria by this process was less than 75 per cent, and the majority of the samples of the filter effluent were stained with iron.

These studies show that ferric sulphate may be used as a coagulant for waters such as the Merrimack River, but that inasmuch as the reactions between the water and the ferric sulphate require much more time than is the case with sulphate of alumina, much larger coagulation basins must be provided or the filters must be operated at correspondingly lower rates. The use of ferric sulphate as a coagulant in water filtration has received little attention, and the different factors which influence its reactions with the dissolved matters in waters are comparatively unknown. That these reactions may be completed in many cases without the addition of alkalies to the water is proved by the excellent results obtained during the summer, when the alkali naturally present in the river water was entirely destroyed and the effluent from the coagulation basin was acid. During later periods, however, the coagulant was not completely decomposed in the water, although apparently there was sufficient alkali present for the purpose. Much more study of the process under a variety of conditions and a thorough investigation of the chemistry of the process are necessary before the conditions under which a uniformly satisfactory coagulation of water by this chemical can be determined.

Owing to its unstable nature, ferric sulphate is not a commercial product, but its production from copperas could be accomplished in the chemical tanks common to any rapid filtration plant without serious difficulty. The ferric sulphate used in these experiments was prepared in the laboratory by the oxidation of commercial copperas with nitric acid with the addition of the theoretical amount of commercial sulphuric acid. Prepared in this manner, the cost of ferric sulphate at Lawrence would be about $34 per ton, but with proper facilities this cost might be reduced to about $27 per ton by the use of commercial sodium nitrate and sulphuric acid instead of nitric acid. On the basis of $34 per ton during the periods when satisfactory coagulation was obtained, the cost for ferric sulphate would have been about $6 per 1,000,000 gals., or about $1.42 per 1,000,000 gals. more than for the amounts of sulphate of alumina and soda ash which previous experience has shown it would have been necessary to use.

Drainage and Irrigation

Management of Irrigation Systems.

Contributed by Robert S. Stockton, M. Am. Soc. C. E., Superintendent of Operation and Maintenance, Western Section, Canadian Pacific Ry., Irrigation Block, Strathmore, Alberta.

It is desired to express here a few of the opinions and ideas resulting from some years of observation and experience in connection with the construction, operation and maintenance of irrigation systems with especial reference to the viewpoint of the management.

Organization.—There are two essential requisites for the proper organization of a force sufficient to operate and maintain a sizeable irrigation system. First, the irrigation manager or superintendent in charge of the system should be an irrigation engineer or an engineer experienced in irrigation work who has good executive ability and character to inspire confidence among the water users; second, the watermaster in charge of the delivery of water and maintenance work and the handling of the ditchriders in a limited district should be a practical man with a sound experience in ditch work and irrigation farming whom all can respect and whose authority will not be questioned. The watermaster is the backbone of the organization, especially where inexperienced ditchriders must be used or where the ditchriders are hired for a portion of the year only, and consequently likely to be changed each season. The watermaster's district is properly laid out when he can drive to any point in it and return to his headquarters in a day, and will usually contain from 10,000 acres to 30,000 acres of irrigable land depending on the layout of the system, size of farm units, location of roads, towns, railway lines, etc.

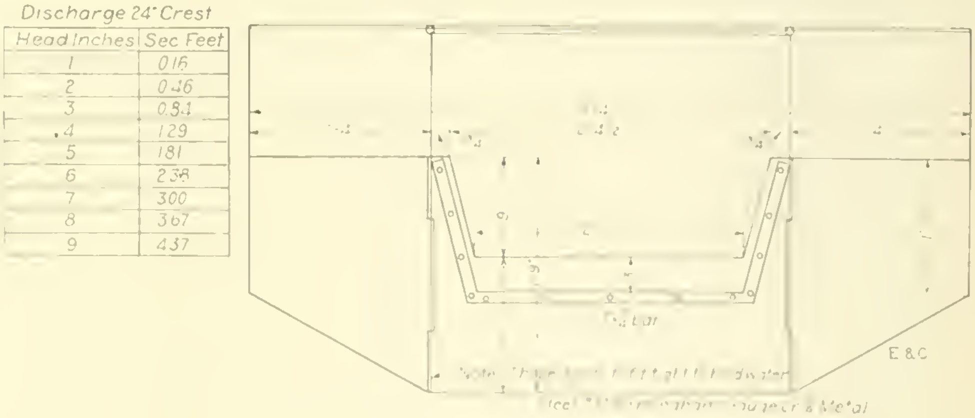
The economical and safe delivery of water depends on having reliable and energetic service from the ditchriders, who are preferably young men. The ditchrider can cover from 20 to 25 miles per day, and generally has from 2,000 acres to 5,000 acres of irrigable land or a length of main canal. The ditchrider should be quartered as near the headgates of his main laterals as possible.

For the position of ditchrider it is desirable to have men who are capable and willing to do hard manual labor, and who can be depended upon under all conditions to get out early in the morning and late at night, or jump in the water, if need be, to attend to their many duties. The ditchrider must be willing to take an intelligent interest in keeping the structures, ditches, and right of way fences in repair at the smallest possible cost, doing as much of this work as possible in connection with his duties in delivering water.

The ditchriders must be able to make clear and accurate reports, and learn to put in small structures of timber and concrete, as well as to handle teams properly on grading work. Not the least part of the ditchriders duty is to learn the best methods of handling water in irrigating the land, the duty of water on various crops, and the best methods of farming. On the Western Section of the Canadian Pacific Ry. Co.'s Irrigation Block, provision is made for four grades, as follows: Assistant Ditchriders employed as required during the season, Ditchriders, who are employed by the year and have houses furnished them and are in charge of a district, District Ditchriders, who have charge of a district and have length of service and knowledge of grading and structure work, Foremen Ditchriders, who have usually been promoted from District Ditchrider after showing special ability in handling men and work, and who would then be in line for promotion to watermaster. The conditions necessitate a large permanent force, since the company is responsible for individual delivery of water, and there are few experienced men available and even common labor is scarce.

The larger the organization, the more important it becomes to educate the force so that promotion becomes feasible. This policy gives each member something to work for and look forward to, and cements the loyalty of the organization.

The Watermasters have also reporting to them the foremen of any grading or repair crews required. Ditchriders are used as foremen as much as possible when the water is not running, as a matter of economy and also to develop experienced men who may be promoted.


Discharge 24" Crest

Head Inches	Sec Feet
1	016
2	046
3	0.84
4	129
5	181
6	238
7	300
8	367
9	437

Fig. 1. Folding Steel Weir for Use of Watermasters and Ditchriders.

The superintendent or irrigation manager will have such engineering assistance as circumstances require. On very large projects an assistant engineer or superintendent will be required over each unit of the work. To illustrate, it may be stated that the above mentioned project has three secondary canals, each serving 100,000 acres more or less of irrigable land and each under an engineer entitled, "Canal Superintendent" for secondary *A*, *B*, or *C* system respectively. On other projects a reservoir system or pumping plant may call for engineering direction and management. Under the responsible engineering heads will be such instrument men and field parties as are necessary. On large irrigation works there is almost always a certain amount of construction work turned over to the operation force on completion of the main features, as it usually is not economy to complete all details of the system until settlement and use has justified further expenditure. This, coupled with betterment work where unforseen conditions have developed, and the renewals where wooden structures have been installed, call for a good engineering organization which should link with the strictly operating force as described. Large projects can afford the services of a hydrographer to make current meter measurements, install gages and meters and accumulate a variety of data in regard to evaporation and seepage losses, duty of water, etc. The hydrographer should be placed in charge of the water records and may be employed in the winter in working up the rating curves and compiling statistics.

The accounting and cost keeping system under the charge of a Chief Clerk will, of course, vary under different management from the elaborate and complete system of the United States Reclamation Service down to the comparatively simple set of books kept on some of the smaller projects. In a general way there must be an accounting of all money spent and received; equipment must be inventoried at intervals, and the cost of work determined: Usually the money must be debited or credited to some fund or appropriation. Another matter is the up-to-date record of the land ownership and payment of maintenance dues or assessments.

It is considered good policy to have the superintendents, watermasters, and ditchriders—in other words, all persons in responsible charge of work keep a daily diary covering briefly all matters of importance in connection with water deliveries, controversies, breaks, damages and repairs, new construction, crops, weather, etc. The significant dates and matters likely to arise in dispute are thus made a matter of record available for use in the courts, if necessary, and are of invaluable assistance to the management in any event. The ditchriders in addition record in their diaries the gage readings, changes in gates or stop planks, the exact time of turning on or off water to any lateral or any water user. This data is copied into the report forms required to be sent into the office An intelligent interest must be taken in writing the diary, and care should be taken to write legibly and concisely.

Telephone System.—A proper telephone service on a big canal system occupies the same relation to running water that it, or the telegraph, does in the operation of the train service. It must be conceded then that the telephone service is of the first importance. The constructing engineers have recognized this fact, and usually nowadays the telephone system is built in advance of the heavy construction work on the canals and ditches so as to benefit by its use during the construction period. It is desirable to have a metallic circuit, and it is economy to build a first-class line wherever its location can be permanently useful to the operation force. As ordinarily arranged, 12 to 15 instruments may be used on one line which is built to serve one unit of the work and connects with a switchboard at headquarters. The number of lines depends on the size and topography of the project. It is desirable to have each watermaster and ditchrider provided with a telephone instrument so arranged that all may be called from the switchboard, and, if possible, so that each watermaster can call his ditchrider direct. This is especially necessary where a central operator is not continuously on duty. The desirability of a separate telephone system covering the irrigation system depends on the necessity for service, both day and night, and with facilities for long conversations without delaying public business, and further, some at least of the canals are likely to be out of reach of public telephone systems. It is very desirable to have each watermaster connected with the public system that serves the community, both as a help in receiving news of a break in a ditch from any private person and also as a convenience to the water users in giving notice of their requirements. In the case of the telephone, true economy consists in having the best installation and most complete service that is practicable. A single break in a large canal might cost thousands of dollars in some cases, and the telephone is the greatest aid in prevention and in ensuring prompt action when a disaster has occurred. Where a telephone line crosses the irrigation system at important points away from the different call stations it is advisable to use iron pole sets or build small booths where the ditchrider may call at a certain hour by appointment or call at any time if he desires to do so. The booth phone is placed under a switch, which is thrown out when the phone is not in use so as not to weight the line. Almost any number of additional instruments can then be used on a line. Suppose such an arrangement reduces the distance a ditchrider must go to give an alarm in case of emergency by 3 or 4 miles, the time thus saved may save the canal; and it must be kept in mind that the cost of the break or interruption to service, from whatever cause, is usually mainly in the crop depending on water, rather than on the cost of repairs to the ditch itself.

Delivery of Water.—The delivery of water by rotation should be insisted on as the governing principle in every irrigation project, as it brings about the best results to the water user, the most economical use of water, and the smallest construction cost. This means that the farmer has time to cultivate as well as irrigate his crop, and that there is more land under irrigation and less damage from seepage. If a man irrigates 2 acres per day with 1 sec. ft. of water he can probably irrigate 5 or 6 acres with 2 sec. ft., and with flood irrigation go over rougher ground.

The watermasters and ditchriders responsible for the delivery must arrange for the impartial and prompt delivery of water on written notice from qualified applicants, in sufficiently large heads to ensure rapid irrigation when needed. For field crops a head of 2 cu. ft. per second should be sufficient for each unit of 80 acres or separate holding of 40 to 80 acres. Larger tracts may be considered as multiples of 80 acres and served accordingly. For best results in most districts in the middle west each owner should be entitled to the use of water for half of the time during each week through the critical portion of the growing season, but good results may be had with less water. The total amount of water used in the season varies with the soil, slope, kind of crop, thoroughness of cultivation, skill of the irrigator, method of irrigation, and natural rainfall. Since the natural rainfall varies from season to season, both in time and amount, and all the conditions differ greatly in different districts or portions of the same district, the duty of water is seen to be a variable amount. The fact that the duty is subject to so many variables makes it doubly important to determine for each district safe values under average conditions, and the minimum amount required under the worst conditions for the guidance of irrigation engineers, managers, and judges empowered to grant water right decrees or legislators concerned with laws affecting the water rights. The water rights of most irrigation systems entitle the owners to a certain amount of water by continuous flow during the irrigation season from certain natural sources. In order that this be used to the best advantage it is not only necessary to deliver the supply by rotation to the water users, but also to conserve by means of storage reservoirs in order to meet the concerted requirements of certain crops and the general heavy irrigation of most crops in the early summer. The number and distribution of the high points in the curve representing the water used for irrigation varies with the crop, district, season, etc., but must be met to ensure economical use of water and satisfactory deliveries. The reservoirs may be part of some independent system and the water traded back and forth as in the Cache La Poudre District in Northern Colorado or be a part of such ditch system in the drainage area considered, but a reservoir system of some kind is necessary on streams where there is land fully to utilize the water. The water user should co-operate with the management of the irrigation system from which water is received by smoothing and grading the land for irrigation and by building suitable head-ditches, field laterals, furrows and checks, so that water may be handled rapidly, effectively and without undue waste. The duty of water can easily be reduced one-half by rough land, poor ditches, and wasteful careless methods. The water user should sufficiently diversify the crops planted so that not all his land requires water at once, and he should arrange his ditches and work so that water can be used effectively both day and night during the period of rotation.

It is believed to be usually best for the ditch company to build the lateral or distributary ditches to deliver to the boundary of each farm unit. This will ensure the consistent location of the ditches and the settlement of right of way matters; it will secure ditches of uniform construction and ample size. This is especially desirable where the holdings are raw lands and perhaps many of the landowners inexperienced in ditch building and the handling of water. Such new settlers have their hands very full for the first few years in breaking up and reclaiming their land and grading, and building the small ditches required on each tract. Under these circumstances, and for obvious reasons it is desirable that the ditch company operate the system and make the water deliveries for the first few years. After habits have been formed, the land brought under irrigation, the property values placed on a sound basis, and tradition established, it is desirable to encourage, and, if the laws and conditions permit, dictate the co-operative handling of at least the smaller units of the system. Many of the older private ditch companies have drifted to co-operative management by the shareholders, who are the land owners under the ditch. The United States Reclamation Act specifically provides for the co-operative management of the system after a majority of the payments have been made. In other cases the ditch company is a carrier of water in main canals and delivers to water users associations covering each main distributory ditch or unit of the system.

Unless the ditch company is organized to make money out of carrying the water it is better to provide in the water contract for a water users association and co-operative handing of the system. If the company makes its money out of carrying the water then it should be reduced to a wholesale business, if possible, by delivering to the main distributing ditches. It is usually the best policy in the first place to put all the profits and costs on to the land itself and provide for co-operative management as above. If this policy is adopted, then while part of the land is unsold it represents just so many shares in the Water Users' Association, which would be voted by the company and the maintenance assessments paid as in the case of any common shareholder. The Water Users' Association is organized under the law, and may have a President, Secretary, Treasurer and three or more Directors who are elected by the shareholders and are entrusted with the management of the system for one year. One or more ditchriders are selected and repairs made as required. The finances are handled by borrowing money at the bank, if necessary, for the disbursements and collecting the assessments at a proper date, with an interest penalty for non-payment large enough to cover the interest paid to the bank. The by-laws provide for a lien against the land for the water assessments and a right to cut off the delivery after a certain date if the assessment is still unpaid. In this way there is no embarrassment or failure to make repairs promptly for lack of funds. The old practice of depending on the farmers to work out their assessments will not work well except on the very smallest units, since there are always delays and trouble. The farmer may work on the ditch, and is encouraged to do so and always given the preference, but puts in his time and is paid regardless of his being a shareholder. He can, of course, take the money earned on the ditch and pay his assessment, but this system leaves the management perfectly free to do work as required and with economy. One great advantage of co-operative handling of the ditch system is that it satisfies the farmers by giving them the means through the annual election of officers to remedy any troubles or mismanagement, even when through lack of interest extravagance has crept in, the shareholders are conscious that it is their own fault and that a remedy exists. Another advantage is that it provides for the most elastic rotation of the water supply to secure the best results to the individual. In some districts where water is scarce and valuable the people have preferred to have the strong, firm, and unbiased hand of the Government Reclamation Service or large ditch company make the distribution on account of the difficulty of getting co-operative action and an impartial management from neighbors, who have been fighting about the water for years. Hence the policy must be worked out to meet existing conditions. But even in these cases it is thought that when the courts have adjudicated the water rights fair results can be had from co-operative management if the by-laws are worded to secure to the management proper authority during the term of office. The following set of by-laws are proposed as embodying the most essential points to be covered:

WATER USERS' ASSOCIATION.

LATERAL DISTRICT NO. -.

BY-LAWS.

(1) **Organization and Purpose**—Organized under Act ...... of ...... for the purpose of receiving water for its shareholders from the Canals and Distributaries of the ...... Project in accordance with their Contract and Water Agreements with the purchasers of land, and further, for the operation and maintenance of the distributary ditches or laterals, waste water ditches, and structures included in Lateral District No. .., and which includes authority for the purchase of supplies, materials and equipment, the hiring of labor, and the conduct of all business in connection therewith. This incorporation is not for profit.

(2) **Name and Ownership** The name of the corporation shall be the "Water Users' Association, Lateral District No. ..." The Lateral District No. .. covers a tract of land as shown on the company's map attached to these By-laws. The membership shall include the owners of all the irrigated land and water rights in the Lateral District No. ... Each member shall be represented by a number of shares equal to the whole number of irrigated acres included in the land owned by him. The company will hold the shares of all unsold lands.

(3) **Management and Board of Directors**—The management shall be vested in a Board of Directors elected from among the members. Each member will vote on all questions in proportion to the number of shares owned by him. The company will vote the shares representing the unsold lands and may designate any employe to vote their stock or take part in the management if elected to the Board. Shares of stock may be voted by letter or by proxy properly endorsed or certified.

(4) **Annual and Other Meetings** An annual general meeting of all stockholders will take place in January for the election of the Board of Directors and making the assessments to provide funds for the association. A notice of this meeting shall be sent by the Secretary to each Stockholder ten days before the meeting. Other general meetings may be called by the Board of Directors or on a petition representing 20% of the stock, but ten days notice must be given the stockholders as with annual meetings. The Board of Directors will hold meetings as often as necessary for the prompt conduct of the business of the association, and at least once a month during the irrigation season.

(5) **Officers and Elections**—Upon the organization of the district five directors shall be elected. If the number representing the year is even two directors shall be elected for two years and three for one year. If the number representing the year is odd three directors shall be elected for two years and two for one year. Thereafter on each odd year three directors shall be elected and on each even year two directors shall be elected to serve two years. After the election is announced by the president or acting officer of the old board the new Board of Directors shall elect one of their number for president, one for secretary, and one for treasurer for the ensuing year. Should a vacancy on the board occur through death or resignation the remaining members shall appoint a stockholder to fill the vacancy. A majority of the board shall constitute a quorum.

(6) **Duties of Board**—It shall be the duty of the Board of Directors to collect the assessments, appoint a district ditchrider and assistants as may be necessary to receive water from the company at the district headgates and distribute it to the water users in accordance with the water agreement of each water user and regulations of the company under the Irrigation Act. The Board of Directors will have control of any property or equipment owned by the association and will purchase such material, supplies and equipment, and hire such labor as may be necessary to keep the distributary ditches, laterals, waste water ditches, and structures of the lateral district in good order and repair, and in shape to deliver water as specified in the Water Agreement of the stockholders or water users. All bills and labor must be paid by check issued by the treasurer on presentation of a voucher made out by the secretary and countersigned by the president. The secretary and treasurer shall keep a set of books in which is recorded all financial transactions of the association, and the secretary will keep the minutes of each meeting of the directors and stockholders of the association. The books of the association shall be audited at the end of each year by two auditors, one appointed by the directors and one by the company.

The Board of Directors shall render a report to the stockholders at the annual meeting closing their year of service. The president shall preside at all meetings of the board or stockholders, when present, otherwise the board shall select one of their number to act as chairman for that meeting.

(7) **Duties of District Ditchrider**—The district ditchrider duly appointed by the Board of Directors is responsible to them alone. The district ditchrider will request water from the company delivered at the headgates indicated on the district map in any quantity up to the maximum amounts due to the district under the prevailing legal duty of water. The Board of Directors, through the district ditchrider must enforce the regulations of the company as to the handling of water and protection of the company's property and must see that the water is not wasted by the individual water user to whom delivery is made, and will have authority to cut off the water supply of any water user who persists in disregarding the regulations or who wastes water. If the Board of Directors through the district ditchrider fails to enforce these regulations or stop the waste of water the company may reduce or stop delivery of water to the lateral district until compliance can be had

The Board of Directors through the district ditchrider shall have power to rotate the water supply among the stockholders in proportion to their shares, so as to give each one, if required, an irrigating head of 2 cu. ft. per sec. for each 80 shares and for a time limited by the necessity of serving all stockholders within the time necessary to secure beneficial results from irrigation applied to the crop or crops then needing water.

No one but the district ditchrider or his authorized assistants can be allowed to change any gate or interfere with the irrigation system in any manner except in an extraordinary emergency to save life or property. The penalty of such interference is the shutting off of water and the right of action for damages.

(8) **Assessments**—At each regular annual meeting the assessment for the ensuing year shall be voted, and shall be sufficient for the regular operation and such repairs, renewals, and betterments as may be decided upon. If emergency work requiring an unusual expenditure is required during the season, the deficit can only be made up by voting a special assessment at a stockholders' meeting, after 10 days' notice of the proposed special assessment. The assessments are levied at a specified rate on each share in the lateral district. Money paid to the treasurer to satisfy assessments must be receipted for on a regular form and paid into a general fund at a selected bank, subject to check by treasurer on vouchers issued by the secretary and countersigned by the president of the board. An arrangement should be made with the selected bank to supply money in excess of funds on hand at a regular rate of interest, and up to a specified amount. Assessments for any year shall be payable half on or before March 1st and half on or before July 1st, and if not paid by these dates shall bear interest at a rate of 2% greater than the rate secured from the selected bank for the association funds advanced. If any stockholder's assessment remains unpaid at the end of the year both he and the company will be notified. The company will then make payment with interest to the association and the amount so paid will become a charge against the land as provided by the company's contract and water agreement. In such case the company will then have the right to vote the shares of the delinquent stockholder and exercise the same rights as he would be entitled to if in good standing. A stockholder in arrears as above will have no voice in the management of the association until his assessments and interest are fully paid and the association notified of this fact by the company.

(9) **Amendments**—These by-laws can only be amended by a two-thirds vote of all the shares of stock in the lateral district at any regular meeting after 10 days' notice of such amendment. Amendments must not conflict in letter or spirit with the contract and water agreement, of the individual with the company, or the regulations of the company under the law.

Maintenance and Operation.—The first consideration of maintenance should be before the construction of the system. Every irrigation district contains examples of poorly built ditches and structures. The amount to be spent on alinement, masonry, structures, etc., at first must of course be governed by the financial limitations of the promoters, as well as by ultimate economy of operation. There is, however, the same reason for considering economy of operation and maintenance versus first cost of construction that there is for discussing these questions in the case of new railroad construction. The analogy between operating a ditch to carry water to specified points and the operation of a railroad to carry passengers' freight to specified points is at once seen, but it is comparatively recently that the same engineering skill has been applied to such work in the west. This was partly due to the fact that canal construction was usually handled by small corporations or associations with limited capital and of course not provided with a trained corps of technical men and partly due to a general lack of men trained or experienced in irrigation engineering. There is now, however, due to the United States Reclamation Service and the large private capital interested in irrigation works, a large number of well trained and expert engineers capable of solving any problems presented, and some monumental works are being constructed. Furthermore, the consensus of opinion and experience is gradually standardizing the suitable types of construction. The maintenance of a canal system during the first few years after construction, and up until the full amount of water has been carried, usually develops many difficulties, even where the best construction has been employed. It is ordinarily during this period that the banks slide and erosion is greatest. After the banks have become compacted and the slides and seeps have been controlled, the maintenance becomes more of a routine matter. Fortunately, the full delivery of water is seldom required at first, and as the difficulties of operating at maximum capacity increase, the difficulties of maintenance will be apt to decrease. Sliding canal banks are most often due to two principal causes, first, the cutting during construction of the toe of the supporting slope on a hillside, and, second, the saturation of the water bank and adjacent strata, thus so increasing the weight and decreasing the friction due to dry or damp material that a slide takes place to restore stable conditions. The remedy must be proposed after studying the cause, and generally consists in lightening the load by removing any unnecessary material, lining the canal at the critical point with some impervious material, inserting covered drains of various sorts, and driving piling to bind together certain strata or take up some of the load. Slides are particularly bad when they occur suddenly, since this is likely to cause the water to break out at a place where the greatest damage from cutting will result. Drifting sands are very hard on the maintenance force as they fill the canal at certain places and are most difficult to stop, since they usually come from above the canal where water cannot readily be used to start some kind of a vegetable growth for a windbreak. If some growth cannot be started resort can be had to protecting fences of boards or brush if it can be obtained. Under some conditions it might be worth while trying ordinary chicken wire netting nailed to the outside and inside of posts set at a determined distance from the canal, so as to form a double fence. Where the canal banks are of sandy material there is a continual loss from the wind and rain wearing them away. The remedy is to get enough soil on top to enable some kind of grass or plant to grow and protect the surface. An actual break in a canal or ditch may be due to many causes, the important ones being the burrowing animals, sliding ground, some defect at a structure, running more water than the canal is built for or more than the ordinary amount, weeds or other debris may clog a structure and cause the water to break over or around and, what is the most dangerous cause in some cases, the sudden flood water from a rain storm or snow melting and breaking into or across the canal. Such flood water washes out railroads and bridges, and unless the cross drainage has been very amply taken care of must also break the ditch. A canal may run for months safely to a certain gage height, when an increase of one-tenth in the depth of the water will enable the water to get into some gopher or rat hole and cause a break. Vegetable growths may cause a lot of work, especially if the velocity of the water is a little too slow for the conditions. In the north an average velocity of 2 ft. to 2½ ft. per second has worked out very well for most soils and without allowing a great deal of growth. In the distributary ditches a uniform flow deep enough to drown certain plants is best. There are cases in mind where a small flow only was required for a few weeks, giving the weeds a chance to nearly close the channel. Later, when a large flow was wanted, it became necessary to mow out the ditch first. Burning out the weeds in the spring is an important matter, not only in order to start with clean banks but also to kill as much weed seed as possible. The operating force do not usually realize what a good agent the ditch water is for the distribution of weed seeds until they have studied the matter from the farmer's standpoint, then they will readily agree that it is a most serious matter. Seeding the canal banks with orchard grass has given good results in the north; other grasses and plants can be used, such as Western Rye Grass, or Red Top, and Clover for a moister soil, but when the banks are very dry and the soil poor it is difficult to grow anything except noxious weeds, while the cost has deterred most managers from doing much work of this kind on their canal system.

CANADIAN PACIFIC RAILWAY
DEPARTMENT OF NATURAL RESOURCES
ALBERTA, CANADA

GAGE READINGS. Week ending...191....

GAGE	Sun.	Mon.	Tues.	Wed.	Thu's.	Fri.	Sat.

District.. ....

Div. No....................................... Ditch Rider

Form A—Cardboard Sheet 5½x9¼ ins. with Ruled Back.

There is a great deal of money spent in rip-rapping around structures and at curves and other points in the canal bank, where some material softer than the average is carried away. Rock, or cement blocks, forms the most permanent rip-rap work, but is usually put in place at a high cost. The essential things in laying rock rip-rap is to start the foundations below grade of the canal so that it will not slide down. Small brush is effective and often cheap. Certain weeds, and especially Russian thistles, are the cheapest and most effective material under many conditions, but if mature should not be used, on account of the seeds being spread over the fields. Manure and straw are often used for rip-rap work, and especially when confined with chicken wire netting is quite effective while it lasts. Sage brush makes good rip-rap material, and does not trouble as a weed on irrigated land.

Canals on sharp curves have a tendency to deposit silt on the inside of a curve and cut on the outside. This can often be remedied by moving the silt to the outside of the curve and grading the bottom low on the inside and high on the outside—analagous to the super elevation of the outer rail on a sharp railroad curve.

After two or three years' operation the largest matter to be solved by the management is frequently the damage or complete loss of good lands due to the appearance of seepage water from the canals and ditches, aided by waste water from excessive irrigation. This rise of the ground water is usually accompanied by alkali, which often collects until the soil cannot even support a growth of native grass. The remedy is drainage, but this sometimes costs more than the land is worth. The damage suits and adjustment of values, or the question of obtaining money for drainage works presents some very knotty problems. The work of the United States Bureau of Soils has shown that seeped and alkali soils can be reclaimed wherever a drainage outlet can be obtained. The important matter at first is to so far as possible prevent these conditions by constructing cut-off drains to intercept the underground flow before it does damage and by providing channels for removing waste water from irrigation, and most important, to insist on a proper delivery of water by rotation under regulations which permit of the supply being cut off if the user does not irrigate and take care of his water without waste. The waste water from irrigation comes largely from lack of ditches, poorly smoothed and graded fields, and the failure of the water user to stay with the work. It is easy to turn the water loose at night, or when some field is finished, instead of arranging for the supply being cut off or providing ditches for the next field well in advance.

This waste of water can only be prevented by a great scarcity of water or a most rigorous and arbitrary regulation. When it comes to such regulation the strong hand of the government or large company is usually much more effective than with the co-operative management. Maintenance work is greatly facilitated by roadways along the main and distributary canals. Such roads enable the ditchrider to use a cart, and thus carry more tools or equipment than if compelled to travel on horseback. The Western Section ditchriders in the Canadian Pacific Irrigation Block are supplied with the following equipment:

Long handled shovel, pitchfork, rubber boots, claw hammer, wire stretchers, pliers, wire, nails, etc., carpenter's level, square, handsaw, brace and six bits, crowbar, monkey wrench, hoe, maul and sacks, notebooks, and a steel measuring weir.

REQUEST TO DELIVER OR STOP DELIVERY OF WATER

CANADIAN PACIFIC RY. ....191....
Dept. Natural Resources, Strathmore, Alta.
Please deliver water to my land at the rate of..............cubic feet per second

From.................................191....until.......................................191....

Please stop the delivery of water to my land on...191....

Canal on Lateral...................................

Turnout.. ....

Land Owner

Farm Unit No...

District.......................................Section..............Tp..............Rge..............

It is required that requests for the delivery of water shall be mailed to the Strathmore Office or handed to the Ditchrider or Water Master so that these cards will be received at the Strathmore Office 48 hours in advance of the time of delivery or 24 hours in advance of the stopping of the delivery of water if this latter date was not given in the first instance.

RECEIVED at..........................by..........................at..........m..........191....

Form B—Post Card with Address Printed on Stamp Side.

Not all these tools and supplies are necessary at any time, but in general the more tools the ditchrider carries the better work he can do. Further, with a good canal road the Watermaster, repair crews and others follow the canal, and it is thus subject to more frequent inspection, and no time is lost in detouring around through the fields. This operating road along the ditch bank or right of way seems to have generally been overlooked by the constructing engineers, and the operating force can usually point out many places where by a slight increase in expense in disposing of waste material or making the canal bank, taking right of way or designing structures, a suitable road could be left at a fraction of the expense required later.

Operation Regulations.—Considerable study has been given to the regulations which should be enforced by the management in order to ensure the successful operation of the irrigation system and encourage the co-operation of the water users and operating force on a large project. After devising several sets of such regulations, the following paragraphs are suggested as covering most of the essential points common to western conditions. Of course, each manager will have to modify and add to these regulations in order to cover the more or less special conditions or laws governing in his case.

CANADIAN PACIFIC RAILWAY—DEPARTMENT OF NATURAL RESOURCES
ENGINEERING BRANCH. WESTERN SECTION, IRRIGATION BLOCK
DITCH RIDER'S WEEKLY REPORT

NAME OF CANAL LATERAL OR LAND OWNER	Water Turned On or Off	DATE	TIME	Measuring Device	Size of Opening	Gauge Reading	HEAD	Discharge Second Feet	REMARKS

Record of Work
For Week Ending 191 Ditch Ride.
Checked by Watermaster

Form C—Paper Sheet 8¼x13¾ ins.

PART 1. REGULATIONS FOR WATER USERS.

Property.—All works constructed or acquired by the Company are the property of the said Company and must be respected. No structure or obstruction shall be placed on the property of the Company and no use made of its property without a written agreement with the proper official representing the Company.

Right of Way.—The right of way for the various canals, ditches, or other works of the Company has been acquired, and the legal description may be had at the recorder's office. The fencing of the right of way will be carried out under the local laws and regulations where it is the policy of the Company to enclose such right of way. The right of way or easement for ditches below a certain size will usually not be fenced. Fencing across the canal or ditch will be avoided except where absolutely necessary, as at highway crossings, in which case easily working gates will be installed and arranged so they can be locked.

Bridges.—Applications from landowners to erect and maintain bridges across waterways will be granted, subject to revocation and subject to reasonable restrictions as to design and location of the bridge.

Co-operation.—It is expected that every landowner or resident of the district will at all times be willing to assist in an emergency to stop a break in the canal or in protecting the structures and property of the Company. In case of danger from floods or other causes the nearest operation official should be promptly notified. The interests of the Company and water users in such matters are identical, as a break in the water service at a critical time is most expensive to all concerned. Co-operation with the Company, and an interest in preventing loss to the Company will result in mutual advantage.

Requests for Water.—(a) A written request for water, on the regular card provided for that purpose, must be delivered or mailed to the ditchrider or watermaster in time to communicate such to the headquarters office at least two full days before the water is wanted.

(b) Similar notice must be given on the day preceding that upon which it is desired that the delivery of water shall be discontinued.

(c) In making application for water the applicant should state his name, farm unit number, lateral, and the head of water he is prepared to handle, as well as the date when he wishes water delivered, and approximate length of time it will be required.

Water Users' Associations.—The Company will provide every facility for the formation and recognition of "Water Users' Associations," comprising consumers located on any common lateral. Standard bylaws and regulations will be printed and supplied, and every assistance given to consumers to effect such organizations on application to the Company's officials.

Whenever a properly organized association so desires it may, under a special agreement, take over the laterals for operation and maintenance under the general direction of the Company. Under this scheme of organization, water would be delivered from the main canal to each lateral headgate to a representative of the water users under such lateral.

Length of Irrigating Season.—Water will be delivered to all qualified applicants between certain dates established by law or by regulation of the Company. The Company may shut down the system during the irrigation season if a break or emergency repairs make this course unavoidable, but in all cases will employ every proper means to complete repairs and resume deliveries at the earliest possible moment.

Delivery in Rotation.—Water will be delivered in rotation to the separate farm units in proportion to the irrigable acreage of each and the total supply available. As far as possible this rotation will be worked out to meet the individual wants and crop requirements of each water user. The rotation will be so arranged that, when required, a good head can be secured for rapid and economical irrigation. This head shall be two second feet for each 80 acres of irrigable land or separate holding of less than that amount.

Measurement of Water.—The water delivered to each separate holding will be measured over a weir, or by some other means.

Interference with Gates.—No one but an authorized officer of the Company or a duly elected representative of a water users' association which has undertaken the operation of any common lateral shall operate, either to open or close any gate, structure, or other irrigation device on the project, nor otherwise interfere with the same in any way, except in emergencies to save property or life.

Note: This regulation will be rigidly enforced. No irrigation project can successfully be administered under any other condition. The control of the water from one end of the system to the other must at all times be solely and absolutely in the hands of the Company. Persons violating these regulations will be promptly prosecuted. Every water consumer is interested in assisting the Company in the enforcement of this regulation.

Pollution of Water.—(a) It is forbidden to disturb, pollute or cause to become impure the water in any of the flumes, canals, or laterals of the Company, or to allow any act to be done by which the same may become polluted or impure.

(b) It is forbidden to erect or cause to be erected or placed within 100 feet of any canal or lateral ditch of the irrigation system of this Company any stable, stockyard, hog house or yard, poultry house or yard, butchering house or yard, water closet, cesspool, manure pile or other structure or enclosure or condition productive of offal or any other refuse injurious to health that may befoul the water in said canal or ditches.

(c) This regulation shall not apply to short ditches terminating on the land of the purchaser not less than 100 feet from the boundary line thereof.

Note: It is necessary for the health of the settlers and for the proper operation of the irrigation system that the canal water be kept clean, and it is expected that all settlers will co-operate with the company in this matter. It will be the duty of the ditchriders to report any case where this regulation is not observed, so that active measures may be taken.

Use of Water.—The water user who makes application for water is expected to put his land in shape for its proper use by constructing a suitable head ditch and system of field ditches, furrows, checks, or borders, to direct the water to beneficial use. The wasting of water will not be permitted. When water is received it must be taken care of day and night until the period is completed. The landowner will be held responsible for any damage caused by a failure to properly control the water. The company reserves the right to shut down the supply if the water is wasted.

Note: The right to the water is based on its beneficial use. The wasting of water waterlogs the land, carries away the fertility, and often causes seepage and loss to lands some distance away.

Farm Management.—It is desired to enlist the hearty co-operation of the farmers in the district in compiling statistics as to the duty of water for different soils and crops, and in collecting data on the crop yield, returns from stock, most profitable crops and system of rotation, etc. Summaries of these returns and information of value to the farmers will be published from time to time as a step towards making the district more prosperous.

The Company regards their irrigation system and the farms under it as a manufacturing plant from which it is desired to obtain the highest returns possible.

PART II. INSTRUCTIONS TO THE OPERATION AND MAINTENANCE FORCE.

Upon the complete organization of the operation and maintenance force, it will be the Company's policy to instruct and educate all its employes in their various duties, in order to ensure the efficiency of the service. Incompetence or tactlessness will not be tolerated, but promotion to greater responsibility and emoluments will automatically follow satisfactory services.

Every employe should realize that he is an important link in the entire chain of organization. The utmost discipline must be enforced, and instruction rapidly carried out, or if impossible to do so, prompt report made to a superior officer.

The Company's success largely depends upon the relations that exist between employes and water users. In their intercourse with the latter, employes should practice courtesy, but firmness. While the Company realizes that it is a difficult matter to please everyone, a little diplomacy and tact will frequently smooth out difficulties before they become acute. The value of employes to the Company will be largely measured by their ability to rigidly enforce the Company's rules, and, at the same time, remain on satisfactory terms with the water users within their districts.

Farm Statistics.—In order to intelligently administer the irrigation system it is desired to obtain the acreage and yield, and water used on each crop, with other data as to the amount of stock carried. The ditchriders will accumulate this information from time to time through the season and record the same on forms supplied by the Company, from which final reports can be made at the end of each season.

REPORT UPON FARM

Of Mr.. ....
(Name) (Address)

Farm Unit No.................. ....District

..................Section.......... Township.................... Range................W. 4th

Tenant or Renter (if any)...................................... ....
(Name) (Address)

Total acres Irrigable................................ No. of days water delivered..............................

Total acres Non-Irrigable............................ Total No. of acre feet used.............................

CROP	No. of Acres	Average Yield	Total	CROP	No. of Acres	Average Yield	Total
Alfalfa				6 Row Barley			
Timothy				2 Row Barley			
Green Feed				Sugar Beets			
Other Grasses				Peas			
Rye Pasture				Roots			
Winter Wheat				Potatoe			
Spring Wheat				Garden			
Oats				Strawberries			
Flax							

Total Acres broken.................................. Total acreage in crop 191.... ....

Acres Summerfallow.................................. Acres Fall Ploughing..............................

EXTENT AND CONDITION OF BUILDINGS AND OTHER IMPROVEMENTS

....

....

....

NUMBER OF LIVE STOCK

......Horses ....Dairy Cattle ....Beef Cattle ....Sheep

...............Swine ....Poultry

GENERAL REMARKS

....

....

....

....
(Date) (Signature)

Form D—Paper Sheet 8½x11 ins., with Quartered Section Diagram on Back.

Field Routine.—During the irrigation season, employes will be started to work at 7 a. m. Canal superintendents, watermasters and ditchriders will endeavor to keep in close touch with all parts of their respective districts. Watermasters will plan their work and trips in order to know that water deliveries are being properly made in accordance with the water agreement and that the policies of the Company are being carried out by the water user. Ditchriders will be instructed by the watermaster as to deliveries and the best method of handling the water. The ditchrider will arrange to patrol all important waterways each day, and especially visit each delivery gate and note that it is undisturbed and properly set to give the required head of water. The ditchrider is held responsible:

(a) For the safety and proper operation and maintenance of his division. The main and secondary canals must always be first in mind, and no effort, day or night, spared to ensure their safety. This means long hours of work at critical times. The people under the Company's system must be educated to instant response to a call for help in an emergency and to give an alarm in case of a break. In any case of break or damage beyond the control of the ditchrider the watermaster and the headquarters office must be notified at once and all steps taken to ensure the prompt opening of waste gates and control of the water. Great care must be exercised to inform the superintendent of every case where an outside party has been co-operative in notifying employes of incipient breaks, or helping in an emergency, as it is desired to send an official letter of thanks in such cases.

(b) When not delivering water the ditchrider should be employed in cleaning and making small repairs on the canals and ditches. The law requires that noxious weeds be cleared from the banks. Weeds and other loose material should be removed from the waterway and piled for burning. Weeds that have not matured may be used to rip-rap banks where cutting has started. In burning rubbish extraordinary precautions should always be observed to prevent damage to the property of the Company or private parties.

(c) All employes, from the highest to the lowest will be held strictly accountable for the orderly neatness and cleanliness of the premises and territory under their control. Equipment must be taken care of and kept in good order. The neat and orderly manner in which an employe conducts his work becomes a qualification as to his fitness for the position he holds. Carelessness and negligence will not be tolerated.

(d) Inspect all structures, the canal banks, gate settings, gages, and deliveries as frequently as possible, and report at once anything out of order that cannot be fixed at the time. Whenever water is turned into a ditch or wasteway or the head increased, the water must be followed through to its destination, and watched until reasonably safe to carry at that depth.

(e) All rising weirs and gate stands should be well oiled and in condition to work easily.

(f) Bridges have been provided at convenient places, and canals or drains must not be crossed by the public at other places. Small ditches and drains may be crossed by fording by arrangement with the superintendent of operation and maintenance. No permission to establish a ford will be given except where physical conditions are such that a ford can be maintained without danger to the Company's property.

(g) No bridges, culverts, boxes, fences, or other structures may be built across any canal, lateral, or drain for which the Company owns right of way without permission of the superintendent of operation and maintenance, and employes finding such work going on will request to be shown the written permission, without which an order will be given to have the work stopped.

(h) Children and others must be kept out of ditches and canals. Bathing in the large canals is dangerous and is forbidden. Cattle must not be allowed to water in ditches or drains except at specified points. Boats, rafts, etc., must not be placed in the canals.

Repairs.—Windstorms damage the ditches in various ways. Waves wash the banks dangerously in places. Brush and weeds are blown into the laterals, and are the cause of breaks and washing of banks. In some sandy localities banks are blown away or the ditch filled up to such an extent as to cause breaks. Dangerous places must be given careful attention, and prompt measures taken to prevent trouble.

A break started in the canal or lateral bank must always be stopped, if possible, without changing or shutting off the water. The delay and expense to the farmers from a shutdown during the irrigation season is so large that the water should be changed or turned out only as a last resort.

When an incipient break has been stopped by shovelling earth, or by using hay manure, weeds, boards, etc., backed by earth or by sacks filled with earth, or by any means that the emergency has required, there remains an important duty that must never be neglected, i. e., the hole through which the water started must be dug back from the temporary dam for several feet, or a trench dug, and then thoroughly puddled in, using some weeds, straw, or similar material in the front face to prevent sloughing off or erosion. Ditchriders should never be without a shovel and a few sacks while on duty. Whenever bad storms or other dangerous conditions develop the ditchrider must get out on his division at night. While this may be disagreeable, it is the only way to make a record for the ditch.

Muskrats, gophers, etc., are a menace to the ditchbanks, and should be destroyed whenever possible. Poisoned grain and traps will be supplied when needed. In some places these animals may be discouraged by destroying the vegetation on which they feed. By keeping the tules and willows well below the outer toe of the bank, the muskrats may keep out of the water bank, if water for their homes exists close by.

Canal superintendents and watermasters will make report in all cases of important repairs or betterment work. If the work is approved, arrangements will then be made to provide the necessary equipment, material and labor, if more than the regular force is required. Ditchriders having breaks or other work they cannot handle without assistance will at once notify the watermaster.

Heavy repair work will be carried on before and after the irrigation season when water can be drained out of the ditches.

Care of Buildings.—Frequent inspection will be made of the various headquarters established by the Company throughout the project, and it will be expected that these will, at all times, present a neat and tidy appearance, inside as well as outside. Buildings must be carefully fireguarded each year where there is danger of grass fires. Employes should realize that they represent the Company in their particular district, and that they will be expected to set an example to the settlers in their vicinity. Gardens must be kept thoroughly weeded, grounds clean and tidy, and outbuildings in first-class order. The above is the Company's policy, and no departure therefrom will be tolerated. Any repairs necessary to buildings that employes cannot undertake themselves must forthwith be reported to headquarters.

Care of Equipment.—The greatest care must be taken of the Company's property. Sheds and outbuildings are provided so that equipment may be kept under cover at all times when not in use. Repairs to equipment must be made without delay, and all losses and breakages promptly reported to headquarters. Employes will be held responsible for the company's property, and carelessness in the treatment of such will be considered sufficient reason for dismissal from the service.

Note: Canal superintendents and watermasters will be charged with the equipment under their care, and will report at the end of the year the condition and location of each article. All equipment must be properly branded or marked. A specific signed statement covering the details of the loss, breakage and wearing out of the equipment and the death of Company stock will accompany the report and represent the difference between the amount charged and the inventory at the time of the report.

The property clerk shall issue no equipment to the field employees until the same has been branded or marked, as above.

All camp equipment entrusted to the care of any of the Company's employes must be turned in to headquarters in proper condition on the closing down of the works. All cooking outfits are to be well washed and cleaned before being packed for storage. Supplies of all kinds which can be made use of are to be properly boxed up and returned to the stores.

When returning tents, roll same up neatly and in shape for shipment. Have size of tent and flies marked plainly on each, stating from whose camp returned. Roll up the tents and flies in separate parcels. If any tents or flies are beyond repair, mark this information on the outside. A complete inventory of such equipment must be returned with the same.

Telephone System.—The Company's telephone system must at all times be kept in the most efficient condition. The successful operation of the irrigation system becomes impossible without a first-class telephone service. Every employe is expected to render assistance in keeping the telephone lines in good order at all times.

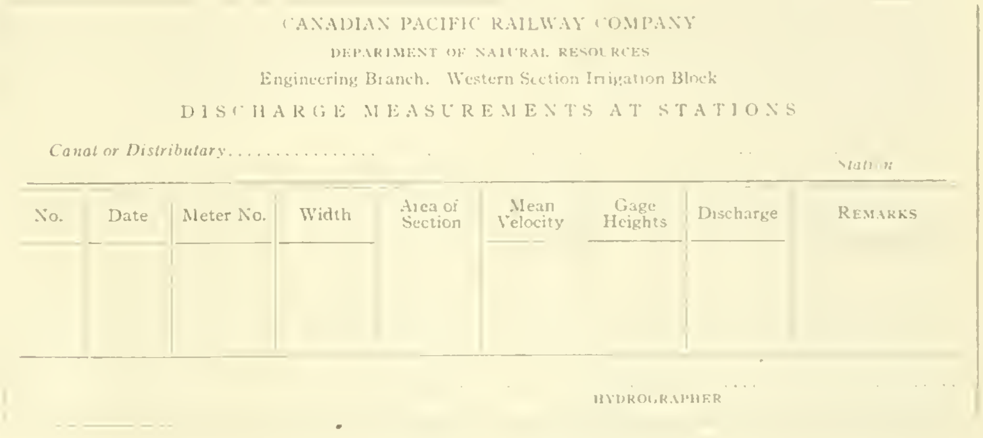
(a) When riding along a telephone line belonging to the Company, watch for trouble. Broken guy wires or braces should be reported, and if breaks are found in the lines make repairs immediately. Take prompt action in the matter when trouble is found. This will shorten the periods when the lines are out of service.

(b) There are three important causes of trouble on telephone lines. These are: (1) Burning out or breaking of wires in fuse blocks. (2) Short circuit of line; and (3) breaking of wires.

(c) In order to protect the telephone instrument from lightning a set of carbons separated by isinglass are placed above the instrument. In case excessive noise interferes with talking remove the carbons and clean the isinglass with a dry cloth and carefully replace in the same position.

(d) When the two wires on a telephone touch this short-circuits the line so that it will be impossible to ring any station, though it is often possible to hear a conversation going on over the line. When wires are discovered touching, separate them and fix them so that they cannot touch again.

(e) If the generator works harder than usual it is a pretty safe indication that there is a short circuit on the line. It is not likely that employes will be able to call anyone in this case, but try to do so nevertheless. In case you are unable to obtain any other station immediately patrol your division in search of the short circuit.



CANADIAN PACIFIC RAILWAY COMPANY
DEPARTMENT OF NATURAL RESOURCES
Engineering Branch. Western Section Irrigation Block
DISCHARGE MEASUREMENTS AT STATIONS

Canal or Distributary................
Station

No.	Date	Meter No.	Width	Area of Section	Mean Velocity	Gage Heights	Discharge	REMARKS

HYDROGRAPHER

Form E—Paper Sheet 8½x11 ins.

(f) When the generator on the telephone turns very easily it is an evidence of a break in the line. When this occurs try to call the next station above and below you in order to find where the break occurs. If you cannot raise the next station, try others above and below you. If no one answers you it is certain that the break is in your own division. Start out immediately and trace the line above and below your station to the next station or until the break is found. If you do not find the break on your own division notify the next man and he will patrol his division.

(g) In case of broken wire on a circuit connect the two pieces of broken wire by baling wire or any other kind of wire that can be had. Scrape the ends of the wires so as to make a good contact. Fasten the wires to the poles so that the wires do not touch each other or the ground. After making this temporary repair notify headquarters immediately.

(h) If you are not called at least once a day by the headquarters office call up that office to ascertain whether the telephone is in good working order. Never leave the receiver off the hook. If it is necessary for you to leave the instrument hang up the receiver instead of laying it down.

(i) In turning the generator crank use a steady motion with one revolution for a short ring and four revolutions for a long ring. Count "One" between rings.

If the above directions are followed carefully by each employe on the line it will seldom happen that the system will be out of service for more than an hour or so.

The Delivery of Water. (a) The company's regulations lay down the notice required on the part of the water users for the delivery of water, and ditchriders and watermasters will give such requests their prompt attention. While the regulations of the company provide for ample notice, if requests can be conveniently filled with less notice it will be the duty of employes to so arrange matters.

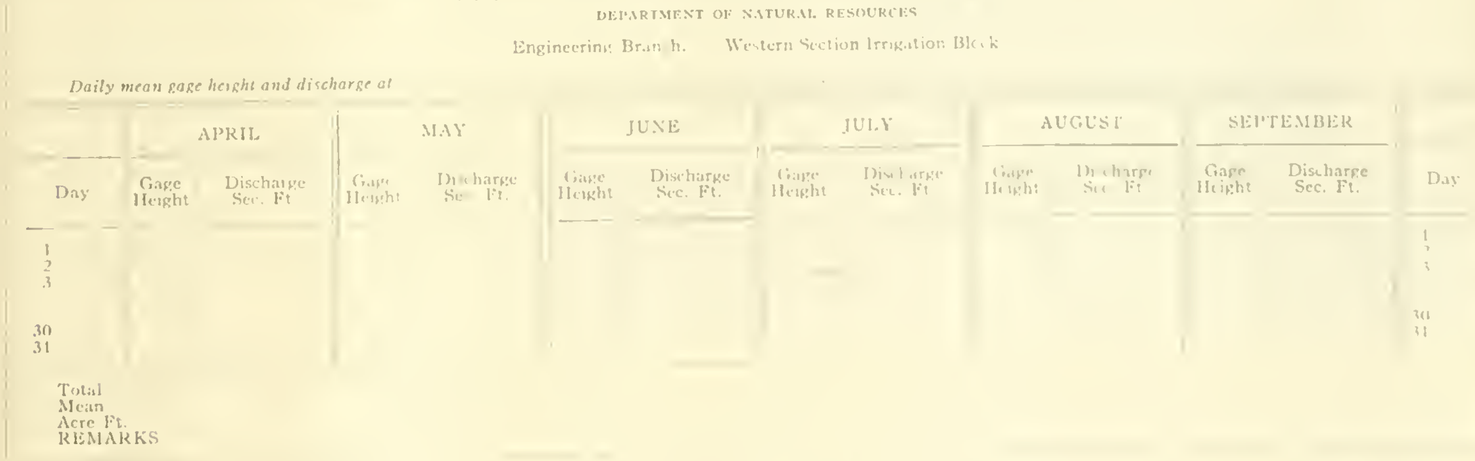
(b) Water is not to be delivered for other than domestic and irrigation purposes, except by special permission of the superintendent.

(c) The distribution of water is the most important of all duties of the ditchriders, and each head of water must be followed to its destination, and measured, and every precaution taken to give uniform service.

(d) A frequent source of complaint on the part of water consumers will be that better services are given their neighbors than they are able to obtain. While, no doubt, in the majority of cases such complaints may not be well founded, employes are warned against showing favoritism towards any particular consumer.

(e) Ditchriders only shall operate any gate or delivery on the project. Gates will be locked to prevent any unauthorized person from raising or lowering them.

(f) Water entering any farm unit must be cared for by the owner throughout the period of the run. The head must not be lowered at night. Consumers must care for the water both day and night and must have a head ditch prepared and ready to receive water by the date water is called for.



CANADIAN PACIFIC RAILWAY COMPANY
DEPARTMENT OF NATURAL RESOURCES
Engineering Branch. Western Section Irrigation Block

Daily mean gage height and discharge at

	APRIL		MAY		JUNE		JULY		AUGUST		SEPTEMBER		
Day	Gage Height	Discharge Sec. Ft.	Gage Height	Discharge Sec. Ft.	Gage Height	Discharge Sec. Ft.	Gage Height	Discharge Sec. Ft.	Gage Height	Discharge Sec. Ft.	Gage Height	Discharge Sec. Ft.	Day
1													1
2													2
3													3
30													30
31													31
Total													
Mean													
Acre Ft.													
REMARKS													

Form F—Paper Sheet 8½x8 ins.

(g) Drainage ditches must not be used as waste ditches except in cases of emergency. Waste water must be cared for on the farm, and if waste water in any quantity runs off it, the amount of water turned on to the farm should be lessened accordingly by the ditchrider after consultation with the water user. Any damage done by waste water entering drains must be repaired immediately by the person responsible for the same. Ditchriders must notify headquarters of such damage, and report failure to repair damage of this sort.

CANADIAN PACIFIC RAILWAY-DEPARTMENT OF NATURAL RESOURCES
ENGINEERING BRANCH. WESTERN SECTION-IRRIGATION BLOCK
.... .Canal, at ....
Date.. ...... Gauge Reading..

Distance from Initial Point	Depth	Depth of Observ.	Time in Seconds	Rev.	Velocity-Ft. Per Second			Area Square Ft.	Mean Depth	Width	DISCHARGE
					At Point	Mean in Vertical	Mean in Section				

Form G—Paper Sheet 5x8 ins.

(h) Telephone reports must be made by ditchriders and gate tenders at such hours as may be designated by the watermaster. Gage readings should be taken so as to be ready in time for these telephone reports. If the ditchrider expects to be absent at report time, previous arrangement should be made with the watermaster. In case of failure to make the daily report the matter will be reported to headquarters.

(i) The measurement of water passing through the irrigation canals of the project is an important duty of the ditchriders and gate tenders. All gages should be read as required, and the amount of water delivered to each farm should be accurately known in order to determine how much water is being used. For the convenience of ditchriders tables have been calculated in order that the proper measurements can be made under the various conditions existing, and will be found at the end of this volume.

CANADIAN PACIFIC RAILWAY CO.
DEPARTMENT OF NATURAL RESOURCES ENGINEERING BRANCH
Western Section Irrigation Block
.... District Farm Unit No ...
Name.....
Address.
Tenant (if any) ...

Year	Date Water Turned On			Date Water Turned Off			Disch. Sec. Ft.	Acre Ft. Used	Crop Irrigated	Acres	Days' Water Used	Duty	REMARKS
	Month	Day	Hour	Month	Day	Hour							

Form H—Cardboard Sheet $8\frac{1}{2}$x11 ins., with Section Diagram on the Back.

Periodical Reports.—Each ditchrider will be required to forward to the watermaster each week a report on forms supplied for the purpose. The basis for the records is, first, gage cards, and second, the pocket diary to be carried by each ditchrider. (1) The gage cards simply show the gage heights at all points on canals and laterals where the hydrographic engineer has established a gage rod and directed measurements to be taken. (2) The pocket diary will contain (a) a brief statement of the operations each day showing how the time was put in, material used, and work accomplished in the division, whether done by the ditchrider or by others working under his direction. (b) A list of people to whom water was delivered, showing location of land by farm unit numbers, the hour water was turned on or shut off or changed in amount with a statement of the amount of water as determined from the delivery weir, or, in case this is out of order by a folding steel weir or by float measurement. The float measurement is made by timing a chip or float through a straight section of the farmer's head ditch and determining the water section, e. g., a float took 60 seconds to travel 100 feet giving roughly a velocity of $1\frac{2}{3}$ feet per second; the channel averaged 3 feet wide and 6 inches deep, which gives a section of $\frac{1}{2}\times3=1\frac{1}{2}$ square feet. The discharge is then equal to the product of cross-section times 8/10 of the surface velocity or in this case $3/2.5/3\ 8/10=2$ cubic feet per second. One cubic foot per second running for 24 hours gives approximately 2 acre feet of water; therefore in the example above the farmer would be receiving about 4 acre feet per day. The diary should mention any other matter of importance connected with the delivery, as, for instance, if the water user disagrees as to the amount of water delivered or makes any threats, or if he has been wasting water, note that you warned him this was against the law and also contrary to the policy of the company, and if persisted in would result in the water being shut off.

(c) A list of new settlers who have gone into occupation upon their land since the last report, giving the location or farm unit number of such land.

(d) Any facts of interest in connection with crops, agricultural operations or general conditions affecting the settlers or settlement.

(e) Breaks in ditches or other trouble necessitating repairs in the system or delays in the delivery of water. Give the cause of trouble, time water was turned off, time water was turned on after repairs, damages, if any, to adjoining land due to flood water or to delays in delivery, work done on repairing break, etc.

(f) Give results of any work done in exterminating muskrats, gophers, etc.

Watermasters will forward gage cards promptly each week through the canal superintendent to the office. At the end of each month they will prepare a monthly report and forward in the same way, together with the ditchriders' weekly reports. The watermaster will keep a daily diary in a book furnished for the purpose covering all matters of importance and serving as a basis for the monthly report. The monthly report should begin with a brief statement of general conditions, including comment or additional facts on any matter in the ditchriders' weekly reports that may be required. Next, should be given a detailed statement of the work accomplished. For each crew should be given the name of the foreman, number of men and teams, material used, earth moved, and the location of each job. In the case of earthwork, get the yardage moved from the canal superintendent, and from the time book figure the cost of each piece of work. A separate set of instructions will be issued covering the use of the time books. Finally, make a statement covering any changes in the regular force, with the reasons for same. A written contract must be signed when teams are hired, giving the rate of pay per hour or per day and any other condition imposed. When laborers are discharged during the month they are given a time check.

Canal superintendents will see that reports of ditchriders and watermasters are promptly transmitted to the headquarters office, and will render a monthly report covering their particular part of the work, and with especial reference to the engineering features. Canal superintendents will keep a careful diary for the protection of the company and themselves, covering all important matters as they occur from day to day. Canal superintendents will expect to assist the watermasters in laying out the water users' ditches for irrigation whenever instrument work is necessary in staking out fills, etc. In some cases the watermaster will be provided with a level and rod, and can do some staking out where this is desirable. During the irrigation season the farmers can only avail themselves of the services of the operating force when this would not interfere with their duties in connection with the delivery of water.

Note: The diaries of all employes will be kept in a uniform book furnished to each, and the books when filled will be filed in the headquarters office as part of the permanent records. The ditchrider's diary will be in pocket size; all others in letter size bound books.

After trying several systems of cost keeping covering operation and maintenance, a system has been adopted which seems to combine simplicity and thoroughness in a satisfactory manner. Any system is dependent on those entrusted to collect and tabulate the data; therefore it is necessary to have a good man in charge of the work. The method under discussion is explained in the following outline, and is designed to facilitate the collection of such data in the time books as will enable the bookkeepers in the office to work up very detailed costs if required, or if not, the charges can be made to the proper appropriation and the time book filed away. Then, if future conditions make a detailed cost statement desirable, recourse can be had to the original time book. The idea is to reduce the work in the field to a simple statement of exactly what each man is doing, the location of the work, and the material used. This relieves the foreman or watermaster on work not sufficiently large to justify a timekeeper, of any difficulties in distribution of the labor cost properly, and concentrates this duty in one or two skilled men at the head office.

Fig. 2. Typical Watermaster's House and Barn.

The basis of the system of cost keeping proposed will be the time book kept on the work by the watermaster, foreman, or other person authorized. The time book will be the fundamental record of all labor and material going into the work, and must be prepared in conformity with these instructions. It should be the duty of canal superintendents to inspect the time books when in the field, and insist on proper records being made.

The time book is ruled and headings printed to facilitate the work of keeping a proper record. The name of each person performing labor is entered at the top of a single page, and his occupation, together with the month and the year, filled in.

Under the heading "Description of Work Performed" must be given the name or number of the structure, ditch, canal, building, etc., with which the work has to do, or its location by quarter section or name of land owner, together with a statement of the nature of the work, i. e., whether the man is employed on carpenter work, concrete work, hand excavation, teaming, scraper holding, or whatever. It should be made plain whether the work is in repairing or renewing an old structure or building a new one. In short, the time book must show definitely the kind of work, the designation of the structure or job, and its location. For each different kind of work performed by a man on any particular day, the number of hours is written in the column headed by the number indicating the day of the month, and opposite the description of the work at the left side of the page. Time should not be divided into fractions of less than one hour.

It is absolutely necessary to show all the time for which payment is to be made, and designates on what work it has been spent. No time will be entered on the payroll for any person that does not appear in a time book with a description of the work performed. The days and hours are added up at the end of each month, the daily or monthly rate entered, and the amount due shown. In this connection there is a page in front of the time book where the names of those having time can be listed under the heading "Detail of Deductions," and the amounts to be deducted for board or supplies shown.

There are also pages headed "Record of Work," under which will be entered each day a brief description of the work done and a statement of the number of teams worked, the number of teams idle, and the amount of horse feed, lumber, cement, steel, nails, or any materials or supplies that have been delivered to the work that day.

The time book must be completed so as to reach the office not later than the last day of each month.

Material or supplies will be obtained through the headquarters office, or purchased locally after obtaining instructions from the office. A written requisition will be demanded for all material and supplies, which will also state the use to which they are to be put. The office will keep a record of these requisitions, and all materials and supplies issued, charging same against the person to whom issued, usually the watermaster. The material and supplies should eventually all be accounted for by the time books and inventories of materials and supplies on hand at the different watermaster's headquarters are turned in. Any excess or deficiency will be promptly investigated and proceeded with until some satisfactory explanation is obtained. If any material, equipment, or supplies are transferred from one watermaster's district to another a transfer should be made out and a copy sent to the office so that one watermaster can be relieved and the other charged with whatever has been transferred. When a man is discharged and a time check sent in to the office, this fact must be noted opposite the amount due in the time book, and care must be taken that all the time shown on the time check is entered in the time book, and the description of the work performed is complete. All persons handling time books will be held personally responsible, and strictly accountable for compliance in this matter. Time will not be divided into fractions of less than one hour, and a daily rate will be understood to mean for 10 hours' labor. Especial pains should be taken to have the time books neat and legible as to writing, and clear as to description of work performed. The proper use of the time book will give the exact cost of each job, however large or small, by simply making proper entry of labor and materials each day in the time book. Great care must be taken to enter up the materials each day as exactly as the labor. Materials will be charged out as requisitioned, and must eventually all be accounted for in some part of the work.

Canal superintendents and watermasters will note that this system relieves them from any trouble in keeping track of costs in their general reports, and their whole duty consists in seeing that every item of labor or material is entered correctly in the time book with a clear and distinct description of the work. In the case of earth work or concrete work the canal superintendent will take such measurements as may be necessary in order to determine the yardage on each separate piece of work. This data will be noted in the time book, and included in the monthly description report. The engineer in charge will see that any work that can be reduced to a unit basis has the proper quantities determined and entered in the time book.

The office force at headquarters will check the time books and make out the payroll, then complete the time books by making up the summary at the end and entering up the labor and material costs on cards or in a separate ledger devoted to a record of cost of work.

The cost cards will be made up first so as to show a distribution of expenditures according to the general headings that must be reported to the board of directors or department. Any sub-division desired can easily be made up. The following list will illustrate one system of distribution:

OPERATING EXPENSES.

Maintenance—
1. Superintendence—salaries of engineers and office force, etc., and cost of repairs to offices.
2. Structures—material and labor, repairs and renewals.
3. Buildings and fixtures—material and labor, repairs and renewals.
4. Waterways- material and labor, repairs and renewals.
5. Telephone lines—material and labor, repairs and renewals.
6. Tools and equipment used for repairs and renewals.
7. Other expenses.

Running expenses
8. Superintendence — salaries, superintendent and office force, cost of stationery.
9. Water distribution—salaries of watermasters and ditchriders.
10. Telephone—salaries of central and cost of batteries.
11. Damage to property.
12. Other expenses.

General expenses—
13. Administration expenses—general offices.
14. Law expenses.
15. Insurance.
16. Taxes.
17. Other expenses.

There will also be a general heading for each appropriation for betterment work or special purpose appropriation which must be accounted for to the department or board of directors.

Under each of the above headings the charges will be distributed under Main Canal, Secondary Canal *A*, Secondary Canal *B*, Secondary Canal *C*, and Secondary *A* Lateral System, Secondary *B* Lateral System, and Secondary *C* Lateral System.

Every piece of work of sufficient size or importance that its separate cost may be required will be so marked in the time book, and will then be entered upon the cost cards under the appropriation heading, as above, from month to month, so as to show on com-

Fig. 3. General Headquarters at Strathmore, Alberta, Western Section, Canadian Pacific Ry. Co.'s Irrigation Block.

pletion the total labor and material cost for that job. Small repairs and maintenance work will be grouped under general headings found most suitable, but distributed under the appropriate canal or lateral system as above.

All time books will be numbered before going into the field, and the cost cards will refer to time book numbers, so that if a complete cost analysis is required of any particular job all the details can at once be found by referring to the time books, which, as the original record, will be found filed away at headquarters. The loss on the running of camps will be charged to R. E. 12, and then for the total cost of canal work will be prorated at the end of the season.

The cost of moving camps will be charged to the new work when this is a sizeable job, otherwise it will be charged to R. E. 12 and prorated at the end of the season for the total feature costs.

In so far as possible equipment repairs will be charged to the work on which the equipment is being used so as to reduce the amount of general expense to be prorated to the feature costs.

It is proposed to make one of the maintenance general accounts a "bank" account for material and supplies purchased and carried in the storehouse or material yard. Such material and supplies will be charged to this account when invoices are passed, provided they cannot be charged directly to the proper appropriation or piece of work. Each month materials and supplies issued and used on the work will be tabulated from the time book charges, and the general ledger bank account credited and relieved of so much of the general charge. The equipment, material, and supplies on hand will be checked up and listed on December 31 each year. Great care will be taken to have all equipment branded or marked as a precaution against loss by carelessness or theft.

Every invoice passed will receive consideration as to its relation to job cost. If possible it will be charged direct. Invoices will be divided into three kinds in this way—first, those for commissary supplies, charged direct to R. E. 12 and offset by credits from the payroll; second, those for equipment, material, and supplies charged to the "bank" account and partially offset by credits from time book charges to different appropriations and accounts under which the equipment material, or supplies have been used; and, third, those invoices charged direct to jobs.

Freight and other expenses will be charged either direct to jobs or to general expenses and prorated against cost of work according to circumstances.

All wages and salaries are stated in full, and board deductions made when the employe is subsisted at a company mess. This greatly reduces the general costs to be prorated to feature costs on the completion of a job over the system of paying so much and board, with consequent charges for subsistence. At the headquarters office there will be kept a general ledger with columns to show the number of voucher or payroll, with its proper name, and date of forwarding, and columns for the amounts distributed to each general appropriation, and totals.

A second ledger will cover the operation and maintenance fund, spread out to show the division to the 17 account numbers mentioned above.

A freight book will be kept to show the way-bill date and number, car number, points between which freight is charged, material carried, invoice number, freight charges, distribution, and cost card number.

An index of accounts will give the name of each firm with the numbers of the invoices passed. Full particulars of any invoice, the number of which is known, can be found on the line corresponding to that number on the general ledger. This book will allow of tracing all the transactions with any firm whose name is known.

The cost cards are lettered to show to which canal system they refer, and are numbered consecutively under each letter. The heading on each card shows under which appropriation it comes under and the feature charged to. The entries on this card show the date, location of the work, and time book or invoice number, and, conversely, the card number is cross indexed onto the time book, requisition, or invoice as these are charged on the cards.

WATER RECORDS.

The general policy followed in devising the forms for use in keeping a record of the water is based on the system of getting all daily reports necessary by telephone and reducing the written reports to a weekly or monthly period. It has been found on large works that it often takes several days for reports to reach the headquarters office, and under these circumstances a daily report is no better than a weekly report, as it is necessary to get certain information by telephone in any event.

The ditchrider carries a pocket size notebook and gage cards in the field. In the book is entered a diary of his work, and all information as to dates and quantities of water delivered, and note of gates changed. On the gage card are entered the regular readings required by the hydrographer in computing the discharges of the various canals and distributaries. This card is shown as Form *A*. The ditchrider is supplied with a folding steel weir as shown in the plan, Fig. 1, and similar to the weir originally designed by Mr. C. D. Howe, Assoc. M. Am. Soc. C. E., of Huntley, Mont. This weir is readily carried in a cart and serves to measure heads of water up to about 4 sec. ft. This weir is used where the delivery is not through a rated box or over a fixed weir. The ditchrider is supplied with cards, Form *B*, to be used by the farmers in making requests to deliver or stop delivering water. At the end of each week the ditchrider makes up from his diary a weekly report on Form *C*, giving the time, date, and gage or amount of water for each change of gates or delivery, with a summary of the week's work. At the end of the season the ditchrider fills out a report on each farm unit, on Form *D*, leaving the water record to be made up in the office.

The hydrographer establishes gaging stations and takes discharge measurements, recording all the measurements at any one station, on Form *E*, and working up the rating curve for that station from these results. The daily gage heights are recorded on Form *F*, and the daily discharge in second feet is then figured from the rating sheet and recorded and the totals computed. The discharge measurements as made in the field are recorded on Form *G*.

At the end of each season a hydrograph is made up for each main canal showing the water carried, the water delivered to lands, and the water spilled at different wasteways, with totals in acre feet. The water records are compiled by filling out Form *H* for each farm unit or water user under the system. (This form should have been printed with a more compact title, giving a larger number of lines for record.) The ditchriders' weekly report form could be reduced to letter size, thus making all the forms of the same size except the gage cards and Form *G* which are intended to be half letter size, and the request cards which are postcard size.

The system of records as outlined has been evolved, after several years' experience, and are satisfactory in practice where reasonably complete data are gathered. The system is easily modified to suit different conditions.

An annual report for the use of the company is an important matter and may also be made to be of great benefit to the headquarters office on the project, and of more value to the company if prepared in a way to constitute a history of each year's operations. This idea has been very thoroughly worked out in the northern division of the United States Reclamation Service and the results are of such far reaching benefit that this viewpoint will no doubt eventually be adopted in all the Reclamation Service reports. In making up the annual report an outline is first prepared covering all the subjects to be discussed. Each chapter or division is then filled in with the data accumulated. Some of the essential matters to be covered are as listed:

1. General Description of the Project.
2. Construction Work.
3. Operation and Maintenance.
4. Lands, Reservations, and Right of Way.
5. Buildings.
6. Equipment.
7. Telephone System.
8. Water Supply and Deliveries.
9. Weather Records.
10. Agricultural Conditions and Crop Census.
11. Organization.
12. Financial Statement and Cost Records.

It may be a big task to make this report so thorough that it will supply the reader with definite facts, and positive information on all important matters, but after the first report has been carefully compiled subsequent ones are comparatively easy to make up.

The report when completed will enable the project manager and the company to answer almost any question with regard to the work promptly, and saves much time in correspondence. Finally, the preparation of the report calls attention to discrepancies in land records and other matters and tends to encourage greater care in accumulating the data. The padding of reports should never be encouraged and the number of reports should be kept to the minimum, consistent with obtaining the information desired, and for which the management is willing to pay.

New Steel and Iron Works Construction in 1913.—There has been an exceptionally large addition to the open-hearth steel capacity during 1913. Practically all of the open-hearth furnaces shown in the following table as built in 1913 were put in operation during the same year:

Built in 1913—Open Hearth.

Operated by—	No. furnaces.	Annual capacity, gross tons.
U. S. Steel Corporation...	14	730,000
Independent companies...	37	2,190,000
Total	51	2,920,000

Under Construction—Open Hearth.

U. S. Steel Corporation...	17	920,000
Independent companies...	29	1,610,000
Total	46	2,530,000

The average capacity of the 37 furnaces built by independent steel companies is raised by the two 200-ton tilting furnaces put in operation by the Lackawanna Steel Co., while the average capacity of the 29 independent furnaces now under construction is increased by the two 200-ton tilting furnaces now being built by the Pennsylvania Steel Co. at its Steelton plant.

Conditions are greatly different as regards the pig iron capaciy, in fact at no time in many years has there been so little capacity under construction or planned as at present. Four coke blast furnaces were put in operation in 1913, with a total capacity of 495,000 gross tons. The capacity of the blast furnaces now under construction is about 430,000 gross tons.

Southern California Association of Members of the American Society of Civil Engineers.—The first annual meeting and dinner of this association was held at the University Club, Los Angeles, January 5, and officers were elected as follows: J. B. Lippincott, president; Charles T. Leeds, first vice-president; George S. Binckley, second vice-president; W. M. Barnard, secretary; Chas. H. Lee, treasurer. The report of the organization committee was given by Mr. Binckley to the effect that the constitution and by-laws of the association, as adopted by the members of the local association at its meeting on December 7, had been approved by the board of directors of the parent society at New York. A committee was appointed to consider the matter of local members of the American Society of Mechanical Engineers joining with them in their meetings. A paper on the Thomas system of reinforcing and three-hinged arch bridges, illustrated by lantern slides, was read before the association by Mr. W. M. Thomas. The secretary of the association has offices at 514 Central building, Los Angeles.

Engineering and Contracting

Devoted to the Economics of Civil Engineering Design
and to Methods and Cost of Construction

olume XLI. CHICAGO, ILL., FEBRUARY 4, 1914. Number 5.

esigning a Structural Steel Plant.

It is only necessary to note the large num-er of manufacturing plants which have failed fter being operated for a short time to ap-reciate the necessity of making a careful tudy of the principles affecting the organiza-on, design and operation of such plants. here is no more dismal sight, especially to he unfortunate investor, than that of a com-aratively new but abandoned manufacturing lant. In many particulars the principles gov-rning the design and operation of a structural teel plant differ but little from those affect-ng other manufacturing enterprises. In mak-ng a recent inspection of a number of steel nills and structural shops we are led to be-ieve that many of these plants are operating nder conditions which do not permit of high-y efficient work. Possibly, both on account of he somewhat rough character of their prod-ct and the fact that the rapid development of he country has offered work for all, some of hese plants have not felt the necessity of in-reasing their efficiency as have other kinds of nanufacturing enterprises. In most cases, owever, the managers realize that their plants re not capable of doing economical work, but or one reason or another have made little rogress toward this end since their construc-ion. No doubt some owners have been satis-ied so long as the plant has continued to make a small profit.

In designing a small structural steel plant the mistake is likely to be made of selecting a site which, even though it may cover enough area, does not possess sufficient width. In some cases it is probable that it would have proved more economical to have abandoned the site as soon as it was realized that the changing character of the work required a different lay-out. Of course the natural tendency would be to crowd new machines into the buildings already erected and to add new buildings wherever convenient. To illustrate this error a well known plant, which is equipped with open-hearth furnaces, rolling mills and structural shops, has been strung out over a very narrow strip of land bordering on a river. In this plant it has been practically impossible to expand laterally, and in the present lay-out the fabricating shop is located between the open-hearth furnaces and the rolling mills. It is necessary therefore to haul the steel billets around the fabricating shop to the rolls and then to bring the rolled shapes back to this shop. Those in charge of the plant of course realize that the lay-out is not an economical one, and, even though they feel the effect of sharp competition, they now find it practically impossible to improve conditions.

The problems affecting the location, design and operation of structural steel plants are so numerous and varied that it is difficult to consider in detail all of them, and some factor, which later may prove to be a very important one, is apt to be given little or no attention. In this issue there appears an article which in a very concise and practical manner enumerates and discusses some of the essential factors governing the location and design of structural steel plants. Many of the principles considered are applicable to other manufacturing plants as well. It is pointed out that the greatest chance for error lies in the selection of the site, and too much attention can hardly be given to such items as market, railway facilities, labor conditions, power supply, distance from the sorrce of raw material, etc. The labor question is always a vital one and usually proves difficult to solve. It is necessary to obtain a nucleus of men who have had previous shop experience, and this often results in placing the new shop near an old established one. A recent example of this is afforded by the proximity of the Dominion Bridge Co.'s plant at Lachine Locks, Canada, and that of the St. Lawrence Bridge Co., which was built to fabricate the new Quebec Bridge. It is also desirable to locate the plant near a large labor market and to place it within easy reach of a rolling mill, as the latter requirement obviates the necessity of carrying a large stock of structural shapes. By means of an "ideal design" the most economical arrangement of buildings and machines is illustrated and described.

What Inducement Does Public Service Commissionership Offer to Engineers?

The query which we have placed as a caption to this discussion is suggested by the quotations in our last issue of remarks by Mr. Newcomb Carlton, vice-president Western Union Telegraph Co., and of Mr. George Gibbs, Consulting Engineer, New York City. Mr. Carlton, while expressing his conviction of the value of engineering skill and knowledge in directing the work of regulation of public utilities, emphasized the fact that technical knowledge coupled with administrative experience were a combination of such worth that there was greater and more remunerative demand for them in other service. Mr. Gibbs' statement may, for the purpose of brief expression, be condensed in three separate statements: (1) As commissioner, the engineer loses touch with details of professional progress and must delegate technical matters to subordinates who are specialists; therefore (2), in a commissioner previous specialized training is of less importance than sound judgment and organizing ability; and (3) engineers as staff employes already have an important part in commission work and often have pay greater than that of commissioners, and commissionership opens up only a small additional field of employment and one of not materially greater honor or remuneration.

The statement of Mr. Carlton is of course absolutely true. The engineer of sound technical knowledge coupled with exceptional administrative ability is in most cases beyond the call of public service commissionership. The same assertion is also true of the lawyer or transportation expert or economist who has similar qualifications. The remuneration and honor attached to public service commissionerships do not perhaps command the service of genius but they do command the service of ability. The question is not: Does the remuneration and honor offered by public service commissionership command administrative genius? The question is: How may we secure the best administrative talent which the remuneration and honor of public service commissionership command? All things are comparative. The salaries paid state public service commissioners are not small, they are considerably greater than a more than ordinary ability in engineering or law regularly commands. Neither is the office without its influence and honor. Perhaps the salaries are not commensurate with the tasks imposed and perhaps more respect should be accorded to the office of public service commissioner, and perhaps both will be increased as time goes on, but now they are as they are and th[illegible] a low standard but a standard higher than common.

The pa[illegible] of public service commissioners [illegible] the service of men of ability. Whether or not the men appointed to be public service commissioners may be thought not to be able has nothing to do with the case. We believe certainly that the most able men available have not always been appointed. Ability, in the sense in which we use it in this statement, involves fitness for the duties to be performed; it involves, besides administrative capacity and experience in organization, a sound general knowledge at least of engineering and other technical matters. As stated succinctly in our quotation last week of Mr. John W. Lieb, vice-president New York Edison Co., most of the problems of public service commissions rest upon the determination of such matters as physical value, functional depreciation, normal wear and tear, obsolescence, suppression, adequacy of equipment, and betterments, and this is an engineering task. It is on these premises that the engineer lays just claim to be one of the best fitted of men to render able service as a public service commissioner.

Mr. Gibbs would set aside this claim insofar as it involves any superior advantage accruing to the engineer as a commissioner because of his specialized engineering training alone. It is difficult to see how this can be allowed. One can readily grant that the engineer commissioner will in time loose intimate touch with advances in the technical details of his profession. It will be admitted also that an engineer of the highest specialized training in his profession, if he is without judicial qualities or lacks the talent for organization or lacks administrative capacity, may be a less useful commissioner than another who can judge, organize and direct wisely but is without specialized engineering training. This last assumption is, however, absurd; an engineer who cannot organize, direct and judge is only in small part an engineer; these faculties are those chiefly demanded of the engineer in his professional work. If he is at all a successful engineer, he possesses these faculties to an extent beyond the ordinary. The faithfulness, sound judgment and organizing ability which Mr Gibbs ranks first of the qualities which a public service commissioner should have are at least as likely to be found in the engineer as in the lawyer or economist. If this is so, his special engineering training or, better, his engineering technical knowledge, is an additional qualification. It may be that his occupation with the broader problems of directing and organizing will get him out of touch with the details of professional progress, but it will not take from him his knowledge of the principles on which this progress is based. He will be a better judge of the work of the engineers of the commission's staff and a better director of the work of these engineers.

As staff employes, Mr. Gibbs considers that engineers now have a status of acknowledged rank. He points out truly that the staff engineer is more highly remunerated often than are the commissioners who employ him. Having the substance, why seek to grasp also the shadow?' seems to be Mr Gibbs' query. Well, why not? Commissionership, being the shadow, is well worth grasping for reasons already stated. There are, however, other reasons which are not without force. Every step forward which the engineer takes in the direction of public affairs is a step forward in public appreciation of the engineering profession. Opening public service commissionerships to engineers will not at the most give many positions to engineers but it will give to engineers the opportunity to demonstrate to the public the many positions in which engineers can serve ably, because of their specialized training. Setting aside any patriotic motive which may have influence with engineers, as it does with other men, we think it is demonstrated by what we have said that public service commissionership has inducements to offer to engineers.

Roads and Streets

Methods of Constructing Wood Block Pavements in European Cities; and a Comparison of Wood Paving in European Countries and the United States.

In our issue of Dec. 31, 1913, we described and illustrated the failure of a wood block pavement due to the washing out of the sand used for cushion and joint filler. According to a paper presented to the tenth annual convention of the American Wood Preservers' Association by S. R. Church, Manager Research Department, Barrett Mfg. Co., such a failure could not occur in London, Paris or Berlin where it is the practice to lay wood block pavements direct on the concrete foundations and fill the joints with bituminous materials. The paper describes the methods of making and laying block pavements in the cities mentioned and contains some conclusions of the author concerning the appearance of European wood block pavements, and a comparison of them with similar pavements in the United States. The paper, slightly condensed, follows:

It has been stated that creosoted wood blocks would make the ideal pavement if the following faults could be eliminated: (1) The tendency to expand, resulting in bulging, and sometimes upheaval of the pavement; dislocation of curbs, etc. (2) Slipperiness. (3) Exudation of oil, or "bleeding. Of all the wood pavements that I saw in London, Paris, Berlin and other cities, none of them exhibited any of the foregoing defects, with the exception of two or three slight bulges noticed in London.

EXTENT OF WOOD PAVING IN ENGLAND.

In 10 of the 28 boroughs constituting the City of London, and comprising the most thickly populated sections of the city, the total mileage of creosoted wood block in 1912 was 121, and of this total 40 miles were in the City of Westminster. The streets of Westminster are important thoroughfares. The traffic is, however, very largely rubber-tired. In fact, the percentage of iron-tired traffic, if known, would probably be surprisingly low. There are a few exceptions, such as the Strand, where there is considerable mixed traffic; but on the whole, the streets of Westminster and those of purely residential boroughs, such as St. Mary Lebone, Kensington and Wandsworth, which together contribute 90 miles to the total of London wood paving, carry a traffic comprising a tremendous number of vehicles, but of a very nondestructive character to pavement. The City of London proper contains only 8½ miles of wood paving. Here the traffic is, of course, intense in all respects.

All of the other English cities have adopted wood paving for some of their best streets. Liverpool has approximately 150,000 sq. yds.; Birkenhead 95,000; Birmingham, Nottingham, Bristol, in fact, all English cities, having over 50,000 population, have wood block streets, and in the majority of cases these are the principal thoroughfares of the town.

APPEARANCE OF WOOD PAVING IN ENGLAND.

In England two classes of wood paving exist, soft and hard. The hardwoods are ordinarily laid untreated, and will not be considered in this paper, except to state that they are going out of favor very rapidly. The creosoted softwood pavements presented, in general, a good appearance, but not equal to the appearance of our best wood block streets. They were not slippery, because the wood is soft enough to permit the pounding into the fiber under traffic of the gravel or coarse sand which is spread over the pavement, and the pieces of grit become permanently embedded in the surface of the wood.

The wear of the blocks is, in general, very uniform, and but few streets are seen where depressions, pot-holes or inequalities in the surface exist, although in the downtown streets of London proper, where traffic is heaviest, there are several exceptions to this general good condition. From a casual observance of the wood block surfaces in London, one accustomed to the appearance of our pavements marvels at the apparent absence of replacements. It requires very close scrutiny of the pavement to find the places where service cuts have been made, and where the pavement has been patched. The great care with which this work is done to produce such good results is in marked contrast with the careless manner in which the blocks are in sometimes thrown back into place after a service cut in one of our city streets.

MANUFACTURE OF WOOD BLOCKS IN ENGLAND.

At the treating plant of one of the most important paving block companies, they are using almost exclusively wood of the class known as "pinus sylvestris," which is sold under various names, such as "Swedish pine," "Baltic pine," "yellow deal," "red deal," etc. This is a most uniform appearing wood, block after block could be examined without any noticeable difference in weight, size of rings, freedom from knots, shakes, etc., nor is there as marked a difference apparent between heart and softwood as in our yellow pine. The manager of this company stated that he has made a study of wood for many years, and has been all over the world, and that in his opinion this Baltic pine is the ideal paving block wood, and to it he ascribes the success of their pavements. The methods in the plant appear to be thorough and well regulated, but not nearly as up-to-date mechanically as our American works. The saws are small; the treating cylinders are small, and there is a great deal of hand labor. Straight pressure treatment is used. The average treatment is about 10 lbs. per cubic foot. The City of London now requires 12 lbs. and the city of Westminster 10 lbs. The oil is coal tar creocote oil, said to be about 1.06 specific gravity. Specifications are very simple, and there is practically no inspection at the plant.

Considering the apparently light treatment, the penetration of the oil is exceedingly good. This company lays most of its own blocks, and a large proportion of its contracts are in London. These blocks are delivered to the street in wagons. The storage capacity for untreated or treated material at the plant is small, so that the wood is not seasoned very long before treatment, nor do the blocks remain in storage after treatment for any length of time. In most of the London paving, the blocks are delivered just about as fast as they are required. They are seldom piled along the sides of the streets.

LAYING WOOD PAVING IN LONDON.

I had an opportunity of carefully observing the construction of a wood pavement in Gracechurch St., London, and was greatly impressed with the good workmanship displayed. The street is in the heart of the old city, and is 34 ft. wide. It was being paved for a distance of about six blocks. The contractors were allowed to shut the street off from traffic for a definite period of time, within which the work must be completed. The old pavement had been entirely removed, and a new cement-concrete foundation 9 ins. deep was put down. This concrete was put down in sections about 25 ft. long, and the full width of the street. It was laid in one course, with a very wet mix, the aggregate being Thames gravel and sand. The concrete surface was finished to an absolutely true grade by means of a wood templet, and without the use of any mortar course. Sometimes a mortar course is used to make the surface smooth. In all cases the concrete is allowed to set six or seven days until perfectly hard, and the blocks are then laid directly on the hard, smooth concrete, without any cushion. This is the universal practice, not only in London, but in Paris and Berlin, and all the engineers with whom I talked said that they were opposed to the use of a sand cushion or a soft cushion of any kind. The blocks were 5 ins. deep by 3 ins. wide by 7 ins. long. They were very uniform in length and exceedingly true in depth. They were laid fairly close, but not rammed. An expansion joint 1½ ins. wide was provided along each curb, and two rows of blocks laid parallel to the curb, with a pitch-filled joint between these rows. The expansion joint along the curb was filled with clay to within ½ in. of the top. Before paving up to projections, such as manholes, pipe valves, etc., the blocks are fitted with great care around all these projections. Over the finished surface after the blocks are laid, a flush coat of hot coal tar pitch, of about 140-145° F. melting point, was poured from hand buckets, and this was immediately squeegeed over the surface with rubber rollers, forcing it into all the joints. This was followed with a thin wash coat of cement grout, and finally a substantial layer of fine, brown, siliceous gravel, free from dust or loam, which was allowed to remain on the surface until pounded into the blocks by traffic.

The foregoing is typical of modern English practice. I have seen statements to the effect that spacing lath are commonly used in the joints, but this practice has been abandoned, except in the case of some hardwood pavements. The spacing strips are never used in London or Paris with softwood blocks. They are, however, used in Berlin. The flush coat of pitch over the surface is the universal practice in England, although not used to any extent elsewhere. Longitudinal expansion joints are always provided, even on heavy traffic streets, but lateral expansion joints are not used. In London 5-in. blocks are used on all important thoroughfares, and no blocks less than 4 ins. deep are ever used.

EXTENT AND APPEARANCE OF CREOSOTED WOOD PAVING IN PARIS.

Creosoted wood block paving in Paris is more noticeable for quantity than it is for quality. Up to the end of 1911, they had 200,000 sq. m., equal to about 280,000 sq. yds., of wood paving. As in England, they have used both hard and soft woods. Hard Australian woods were used to quite a large extent, but have not given satisfactory results. The report of the Chief Engineer of Bridges and Streets of Paris, at the London Road Congress, stated that the average life of the hardwood paving was six years. The appearance of the creosoted pine pavements in Paris varies from good to exceedingly bad. There are many streets where the blocks are decayed. There are also many very rough streets, and on the whole, they are distinctly inferior to the wood block streets in any of our American cities.

MANUFACTURE OF CREOSOTED PAVING BLOCKS IN PARIS.

I was able to inspect the municipal wood block plant, through the courtesy of M. de Puligny. This is a very large and well ordered plant, and the first thing that attracts attention is the very high lumber piles, which approximate 40 ft. The sticks are run up into these piles direct from the cars by means of an electric tiering machine. A large proportion of the lumber is kept under glass roofs. There is also a very large storage yard for treated blocks. A new electrically-driven machine, which saws 16 blocks at once, is a feature of the plant. The sticks are fed automatically to the saws, and the blocks are discharged automatically with the grain up, to the inspection table, from whence they are loaded by hand into very small iron treating cars. Five or six of these cars full of blocks are run under the creosote oil tank, where each car is filled up by gravity with oil at a temperature of 80° C. After remaining in this bath of oil for 20 minutes, the oil is drawn off from the bottom of the car, and the blocks taken to the storage yard. The average absorption by this treatment is 3 lbs. per cubic foot. Naturally, the penetration is very slight. It is not surprising that the blocks decay before they are worn out, but at first sight it is very surprising that they do not bulge and buckle continually.

An interesting feature of the Paris plant is the machines for shaving old blocks for relaying. Blocks taken up from the street, if not too badly worn or decayed, are brought to the plant and the edges trimmed on these machines. It is sometimes done with portable machines on the streets. Many of the blocks being repaired in this manner for relaying showed hardly any evidence of treatment, and some were partially decayed. The Paris engineers freely admit that their blocks have been very insufficiently treated, and that the results have been unsatisfactory, and, in fact, they are now erecting a new plant for pressure treatment by means of a process developed by M. Labordere. As he has discussed this process in a recent paper before the Sixth International Congress of Testing Materials, I will not describe it. M. Labordere told me that it is their intention to use, in connection with this process, a mixture of coal tar pitch and creosote oil of not less than 1.08 specific gravity. They have been using for the dipping treatment an oil of about 1.04 specific gravity.

The woods used in Paris for creosoting treatment are Baltic pine, and pinus martimas, a native pine largely used at present. This is a much wider grained wood than the Baltic pine, and less uniform in character. They make an interesting distinction between the resinous or gummy wood from the lower part of the trunk, and the wood from the upper part of the trunk, using the former for heavy traffic streets and the latter for ordinary work. I do not know to what extent they are able to carry this out in practice.

LAYING WOOD BLOCK PAVEMENT IN PARIS.

I saw two or three wood block streets in course of construction. Five-inch blocks were being put down, and in general, the method is much the same as in London. The concrete foundation is very similar in appearance, and the absence of any soft cushion especially noteworthy. Expansion joints 1½ ins. wide along the curbs are filled with sand. Two rows of blocks are laid parallel to the curb with 1-in. between the rows, a peculiar type of sectional expansion joint being used, consisting of a bituminous paper box about 6 ins. long, containing hollow paper cylinders. This keeps dirt, etc., out of the joint, and crushes readily under pressure. Transverse joints about 100 ft. apart are also filled with these bituminous boxes. On another street a collapsible metal expansion joint is used. Instead of pitch grouting, it is common practice in Paris to flush the surface with cement grout, which fills the joints that are open. The work did not seem to be as well organized or as rapidly carried on, nor did the surface of the finished pavement present as uniform an appearance as the London work already described.

In another street I saw 5-in. Swedish pine blocks being laid. The grade was at least 2½ per cent. In this case the surface was flushed with coal tar pitch poured by hand, and forced into the joints by the use of hot smoothing irons. This was followed by a heavy covering of coarse sand. I was told that this method of filling joints was not as common in Paris as the cement grouting method.

WOOD BLOCK PAVING IN BERLIN

There is not much wood block paving in Berlin, but a condition which seems rather remarkable exists, in that on streets paved with asphalt, where the grade becomes too steep and the asphalt will prove unduly slippery, wood block is used. Most of the approaches to bridges are paved with creosoted wood block.

Berlin has probably the finest sheet asphalt paving in the world. The wood paving is rough and comparatively noisy. About the only thing of special interest noted in Berlin was that a new wood block plant has been erected where the Rueping process will be used for treating Swedish pine similar to that used in England. I was shown blocks treated by this process, which appeared to have very good penetration with a minimum use of oil.

CONCLUSIONS.

In the beginning of this paper I stated that European wood pavements did not bleed, expand or buckle, and were not slippery. Let us inquire briefly into the reasons, and see what we have to learn from their methods.

Conditions are so entirely different that I do not see how any direct comparison can be made. In the first place, as to slipperiness, I have already explained that the nonslipperiness of their pavements is due to the use of softer wood. As to bleeding, they seldom have any weather in London hot enough to cause bleeding. In Paris there is not enough oil in the blocks to cause bleeding. The climate also undoubtedly plays a very important part in the question of expansion and contraction. It is agreed that the sudden expansion that sometimes takes place in our wood pavements is due to a rapid absorption of moisture, usually after the pavements have been unduly dry. This can hardly occur in England, where they have a little rain all the time, very seldom any torrential downpours or any long continued dry spells. The average humidity is high. It is a question whether the pavements ever get dried out to anything like the extent that ours do. I should say that generally the blocks are in a maximum condition of expansion. The same must be true in Paris, or the inadequately treated blocks in that city would certainly bulge. On the other hand, it is well worth considering if the kind of wood that they use has something to do with the lack of expansion troubles. It is manifestly a very different wood from our longleaf yellow pine. As I have already stated, it does not present as attractive an appearance, and it is probably not so durable. It may be, however, that with shortleaf pine or tamarack or Douglas fir, we could obtain results more nearly approaching the European results by following their practice more closely with regard to treatment and methods of laying.

I believe we can afford to give careful consideration to the question of lighter treatment, but I do not believe that because 10 or 12 lbs. is ample in London with the wood they are using and the climate that there exists, that amount would be ample for our woods in our climate

The points that I would especially emphasize for your consideration are: (1) The great care used in the preparation of the concrete foundation and, in fact, in the whole workmanship of the pavement; (2) the general use of deeper blocks; (3) the universal practice (in England) of sealing the joints of the pavement with coal tar pitch, so as to prevent the entrance of moisture

I would also emphasize that the pavements in Europe (this does not apply only to wood), are laid and maintained by workmen who have more conscience, or at least, take greater pride in their work than we find on the average job in our country. There is less inspection on the part of the city of the operations of making the blocks and laying them, but I got a very strong impression that the manufacturers are taking no chances of having their material condemned on account of carelessness or bad workmanship

In conclusion, I want to repeat that in my judgment, none of the European wood paving surpasses our best wood streets in appearance.

Cost of Surveying Built-up Towns.—According to a paper read at the annual meeting of the Oklahoma Engineering Society, by Mr. H. V. Hinckley, consulting engineer of Oklahoma City, many towns of 5,000 population had been built up by white trespassers in Indian Territory before Congress officially allowed them in that territory. Most of these towns were built up without survey. The consequent necessary survey and platting cost $2 to $6 per acre, compared with a cost of 50 to $1.10 per acre for surveying a town on a prairie, in advance of settlement.

Some Road Crusts of Low Construction Cost Suitable for Roads in the Southern States.

In a paper presented at the recent convention of the American Association for the Advancement of Science by Major W. W. Crosby, Chief Engineer, Maryland Geological and Economical Survey, some types of road crusts that can often be constructed at low first cost and that are suitable for roads in the South, are described. Descriptions of the materials used in some of these road crusts which are not as well known as others, and average costs per square yard of such surfaces are given in the following excerpts from the paper:

Burnt Clay Roads.—Burnt clay roads as developed by the U. S. Office of Public Roads are, of course, limited to localities where conditions are favorable to their construction and where the traffic will permit their satisfactory maintenance with reasonable economy. Their selection should be always based upon careful consideration along scientific lines and of all the circumstances. Their degree of permanence is relatively small. Burnt clay road crusts vary in cost, but under ordinary conditions where they are possible it is estimated their cost would be about 15 cts. per square yard.

Marl Roads.—Marl, being an earthy mixture of shells, clay, sand, etc., very much like (except that the shells in marl are usually much smaller) the dredged shells referred to later under Shell Roads, may almost be considered as included under the remarks on the latter. Marl roads are relatively cheap in first cost and high in maintenance costs, but frequently are worth consideration under peculiar local conditions.

Shell Roads.—Shell roads crusts will be considered under two heads, "Shell Roads" and "Shell Macadam." In many localities through the South a mixture of shells with sand, silt, etc., is obtained by dredging in the streams and is frequently available at low cost. This mixture, when containing a sufficient proportion of shells and properly applied, furnishes frequently a very satisfactory road crust for light traffic at a first cost of, say, 30 cts. per square yard. It is, however, liable to loosening in dry weather unless oiled and readily becomes "horse pathed" under periodic horse-drawn traffic in one direction.

Shell macadam is built, as in the case of broken stone or gravel macadam, from the shells themselves—obtainable from shell heaps at the canning factories or elsewhere—spread, compacted, and then the voids filled with sand or similar material by the aid of water or other liquids. Using the shells in this way, just as if they were broken stone, secures the greatest value from them and the shell macadam so formed is light, strong, reasonably resistant, and particularly valuable for moderate traffic on sandy roads. The addition of pitch by the penetration method is also successfully made to shell macadam and it is easily practicable to construct and maintain a pitch (or bituminous) carpet on the top of shell macadam, which will add greatly to its field. In the cases where pitch is so used, the finer mineral material to go with the pitch should perferably be clean pea gravel or stone chips rather than fine sand. Shell macadam has a pleasant resiliency not possessed by either gravel or broken stone macadam. Its first cost (waterbound)—depending of course on the price of the shells—will vary between 25 cts. and 50 cts. per square yard.

Slag Roads.—Again we may separate the consideration into "Slag Roads" and "Slag Macadam." Basic slag, direct from the pile or dump, because of its heterogenous character, usually gives unsatisfactory results on the road though it possesses the valuable quality of setting up like cement and forming a concrete-like road-crust capable of supporting heavy loads. But no matter how carefully spread and rolled, its final compaction after being subjected to traffic and weathering is generally very uneven. The fine portions, the cellular or honeycomb-like fragments, the massive fragments and the different size particles all give different results. Hence the use of slag direct from the pile has been practically abandoned for road crusts.

If now the slag can, as is sometimes the case, be obtained broken to size and screened as is broken stone, it then becomes possible to use such slag for making macadam, frequently to great advantage because of its cheapness and because of its capacity for strongly cementing together to form a road crust highly resistant to displacement under heavy loads and to ravelling in dry weather. Waterbound slag macadam is inclined to dustiness, but this dust is obliterated by the first heavy rain, as in the case of shell macadam, the dust re-cementing when sufficient water is added to it. The first cost of slag macadam may be as low as 50 cts. per square yard. The use of pitch by the penetration method with slag macadam presents no unusual points for consideration here.

The above remarks concerning slag are intended to cover only that known as "basic" slag. "Acid" slags are not fit for road crusts except in rare cases and when pitch is used in connection with them, as they are vitreous, too brittle, and lack cementing qualities. They may rarely be used in the first course or foundation as a substitute for broken stone or gravel.

Transporting Pipe Line Material Across a Desert by Motor Trucks.

(Staff Article.)

In constructing an oil pipe line between the Midway Oil Fields and San Pedro, California, some 35,000 tons of pipe line material and material for pumping stations were successfully hauled across desert and over mountain trails by motor trucks. There were altogether 18 trucks operated by the contractor, the Auto Delivery Co., of San Francisco, Cal., and these trucks made an aggregate of 700,000-ton miles. White trucks were used, and the performance of these trucks and some of the operating methods furnish information of general interest.

The route crossed a desert for 40 miles. The roads were very sandy and soft, the sand being in some places so loose and deep that the front axles of the trucks dragged on the ground. In several places it was necessary to haul material into the mountains where there were no roads. Sage brush was gathered from the desert and eucalyptus branches were cut in great quantities and spread crosswise of the trail along an arroya. This mixture, and a certain percentage of straw, was sprinkled with sand, making a very good motor truck road over which no difficulty was experienced. Furthermore, it was built at a small expense. Bridges over small streams and ravines had to be built at several places.

In one spot where a road had been built along the side of a stream, the bank caved in and a truck landed upside down in the bed of the stream. After being placed "on its feet" the truck was driven down the stream. This experience demonstrated that the river-bed made a better road than the one which had been used and the road was therefore abondoned. By cutting the bank away so that the trucks could get down to the bed of the river, they were enabled to travel nine miles down the stream. The bottom was semi-quicksand, with water running over it, and it was kept comparatively hard. The truck drivers soon discovered, however, that if they stopped their trucks anywhere in those nine miles the truck promptly settled in the sand. On one occasion a truck went down to the axles before the driver could build a levee around it to prevent the water from undermining the wheels. As long as they were kept moving the bed of the river made a very good roadbed.

It was not an uncommon thing for the trucks to lose traction on the desert roads. The wheels would spin in the sand and dig down until the axles rested on the ground. For such emergencies the trucks carried two 5-in. pipes about 15 ft. long, with the ends flattened out and bent over. The flat end of one of these pipes was placed under the rear wheels and between the dual tires. Another scheme that was used to improve traction on the desert road was the dragging of a 10-ft. railroad rail behind a truck every other day. This filled up ruts and kept the desert road in fairly good condition.

In places the grades were as steep as 32 per cent, and summits having altitudes as high as 8,500 ft. above sea level were features of the route. On the steepest grades it was necessary to use air pressure on the gasoline tanks to make the fuel flow to the carburetors. The thermometer often registered 122° F. in the shade. The wind frequently blew about 30 miles an hour, so that the crews rode in a hot blast and a cloud of dust so thick they could hardly see 20 ft. in front of them at any time.

The great heat and the soft roads made it impossible for the best Missouri mules to haul more than 1,000 lbs. to the animal and these animals, hauling 1,000 lbs., travel only about 1½ miles in one hour (16 miles being a day's work for 10 mules hauling 5 tons). In places remote from the railroad, alfalfa hay sells at $35 a ton, making the cost of feeding a mule not less than $1 a day. Furthermore, the death rate among the mules, due to heat, was excessive.

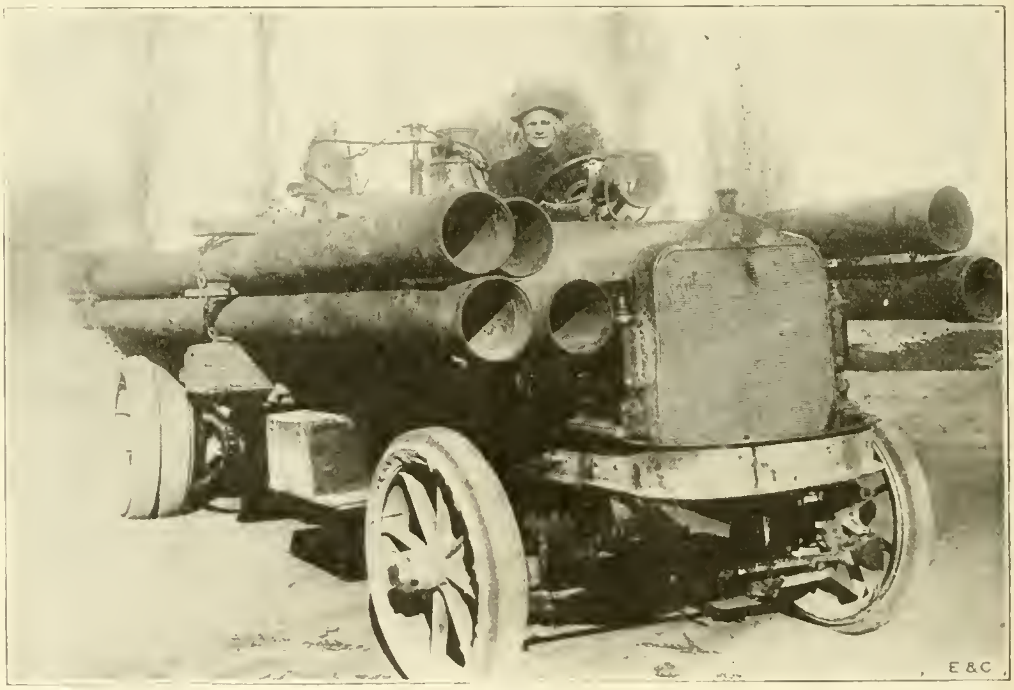


Fig. 1. Motor Truck Loaded With Oil-Line Pipe for Transportation Across a California Desert.

The principal material hauled was 8-in. screw-joint line pipe. As shown in the view of one of the loaded trucks, Fig. 1, the pipes were lashed to the sides of the car so that the drivers could quickly drop them along the roadside. Considerable Portland cement and flat steel plates for constructing oil tanks and various kinds of material for power house construction were transported in the usual manner on the platform bodies of the cars. Extra gasoline, oil and water were carried in 10-gal. milk cans. Blankets and enough food and water to last two days formed a part of each load. This equipment was often necessary. Although the temperature ran as high as 122° F. in the shade in daytime, it was very cold at night on account of the high altitudes which were reached in the mountains on nearly all trips.

The roads were so soft and the operating conditions so difficult that lower gears were installed in all the trucks. The gear ratio between the engine and the rear wheels was about 50 per cent lower than used for normal working conditions. The average time for 40 miles was only about 8 miles an hour. Five of the trucks which had each covered over 100,000 miles before being placed on this job, made good records by making a trip every day along with the newer trucks.

One of the older trucks was driven by an Indian who lived on the desert. He learned to drive shortly after work on the contract had commenced, and he drove 60 round trips of 80 to 110 miles each without a single mishap or loss of a minute from the schedule which had been mapped out for his truck. He was paid 50 cts. per hour and he drove 16 hours a day every day for 60 days. No other driver was able to stand the heat or fatigue as well as this native Indian.

Of the ten trucks hauling material out of Lancaster, the contractor planned to have two laid up all the time for adjustments, repairs, lubrication, overhauling, etc. Eighty per cent of the equipment was on the road all the time and 20 per cent was in the temporary garage which was built at Lancaster to maintain the trucks on this job. The cost of operating the trucks was 50 per cent more than under normal conditions, but as the roads were generally soft the tire costs were less than average. One of the facts brought out in this work is that tires which ordinarily wear out in 8,000 or 10,000 miles on city streets, last three times as long on these soft desert roads. Inasmuch as the earnings of the truck under these conditions were more than double what they would be under normal conditions, the additional operating expenses being only 50 per cent greater, the profit in the business was very much greater than to be found in normal work.

Summary of Irrigation Results, June 30, 1913—According to a statement by the U. S. Reclamation Service, there are now completed or under way 28 Government irrigation projects, comprising a total of 3,191,048 acres of irrigable lands. At the end of 1912 a total number of 641,397 acres were actually irrigated. About 38,767 acres of public lands for which water is available are unentered, or about 465 irrigable farms. This acreage is scattered over 10 projects and as a rule is the least desirable land. Crops produced in 1912 were valued at $14,479,368, an average of $25 per acre. The total area which will be reclaimed by present government works is 3,191,048 acres. The annual crop returns are estimated at about $80,000,000.

Methods and Cost of Constructing a Sand-Gumbo Road in Nebraska.

Among the 41 object-lesson roads constructed by the United States Office of Public Roads during the fiscal year 1912-1913 was the sand-gumbo road built near Columbus, Nebr. In Bulletin No. 53 issued by the U. S. Department of Agriculture the methods and costs of constructing this road are given as follows:

On August 19, 1912, work was resumed on the construction of the sand-gumbo road extending northwest from the Platte River toward Columbus, Neb. A section of road 3,002 ft. long was added to the section constructed during the fiscal year 1912. The roadbed was graded to a width of 32 ft. in cuts and 24 ft. on fills. A sand-gumbo surface 16 ft. wide was constructed having an area of 5,337 sq. yds. The section was completed on Sept. 4, 1912.

Earthwork.—The maximum grade was reduced from 13.2 per cent to 4.4 per cent. The adjacent land is level and the soil is sandy. The earth was loosened with plows and hauled in drag, Fresno and wheeled scrapers. The average haul was 160 ft. and the maximum haul was 350 ft. In the excavation 760 cu. yds. of earth were moved and the maximum cut was 1.3 ft. and the maximum fill 2.7 ft.

The construction outfit consisted of 4 drag scrapers, 2 Fresno scrapers, 1 wheeled scraper, one 8-horse road machine, 1 steel road drag, 1 plow, 1 disk harrow, 1 spike harrow and the necessary hand tools. Labor cost $2 and teams $4 per 10-hour day. Table I gives the cost of the earthwork.

TABLE I.—COST OF EARTHWORK FOR SAND-GUMBO ROAD IN NEBRASKA.

Item—	Amount.	Unit cost per cu. yd.	Unit cost sq. yd. wearing surface.
760 cu. yds. excavation...	$120.00	$0.158	$0.0225
Shoulders and ditches....	46.40	....	0.0038
5,337 sq. yds. shaping subgrade	28.20	....	0.0052
Miscellaneous	1.40	....	0.0002
Superintendence	4.20	....	0.0008
Total	$200.20	....	$0.0375

Materials and Methods Used in Making Wearing Surface.—The surfacing material consisted of a good quality of black gumbo and sharp clean sand. The gumbo was spread to a depth of 7½ ins. and the sand to a depth of 6 ins., both measured loose. The two materials were then mixed by means of plows and harrows and shaped with a steel drag and a road machine. The compacted depth of the finished surface was 8 ins. and the crown was ¾ in per foot. In this work 1,165 cu. yds. of gumbo and 890 cu. yds. of sand were used. The gumbo was hauled approximately 2 miles in slat bottom dump wagons having a capacity of 1 cu. yd. The sand was hauled a distance of 4,000 ft. in the same wagons. The cost of the wearing surface was as given in Table II.

TABLE II.—COST OF SAND-GUMBO WEARING SURFACE FOR A ROAD IN NEBRASKA.

Item—	Amount	Unit cost per cu. yd.	Unit cost sq. yd. wearing surface
Purchase of gumbo pit..	$ 41.35	$0.035	$0.008
Loading gumbo.........	180.40	0.155	0.034
Hauling gumbo.........	698.80	0.600	0.131
Spreading gumbo........	34.00	0.029	0.006
Loading sand...........	93.60	0.105	0.018
Hauling sand...........	299.00	0.336	0.056
Spreading sand.........	10.60	0.012	0.002
Mixing sand and gumbo.	37.20	0.018	0.007
Shaping	4.00	. . .	0.002
Rolling	13.60	....	0.004
Miscellaneous	12.60	.	0.002
Superintendence	37.80		0.007
Total	$1,162.95	....	$0.2745

The following is a summary of the cost

	Amount	Unit cost per sq. yd wearing surface
Earth work.......	$ 200.20	$0.0375
Wearing surface..	1,162.95	0.2745
Total	$1,663.15	$0.3120

Highway Engineering by District Highway Engineers as Managers.

Want of experienced and skillful direction of road work is one of the commonest of complaints in many western states. A suggested remedy for this deficiency is outlined in a paper recently presented before the Kansas Engineering Society by State Engineer W. S. Gearhart. The following extracts summarize Mr. Gearhart's ideas:

In sparsely settled states like Kansas, Oklahoma, Nebraska, the Dakotas, and Minnesota, it is not practicable to employ a County Engineer in every county for the county unit is too small. Except in the big, wealthy, populous counties there is not enough work at the present time to keep an engineer employed throughout the year. If, however, two or more counties were combined to employ a district engineer it would be possible to pay a sufficient salary to get the services of a thoroughly competent man and it would provide an economical, practical system and would be in strict harmony with the very best systems of road management adopted by the states ranking first in highway improvement.

If the counties were combined into districts with due consideration of the railroad facilities it would be practical to include as many as six counties in one district in the western part of the state of Kansas, and provision could be made to reduce the size of these districts to meet future needs. I have studied the matter carefully and am convinced that 45 districts as shown by Fig. 1, would answer every requirement, now, and probably for the next six or eight years.

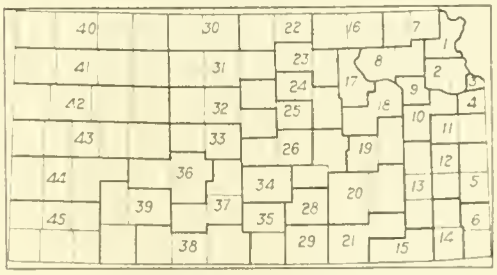


Fig. 1. Map of Kansas Showing Highway Engineer or Manager Districts.

The district highway engineers or managers should be appointed on their merits only, by the county commissioners for a term of not less than four years, but there should be a simple, direct means of removing them from office, at any time, for cause. No person should be eligible for appointment until he had secured a certificate of approval from a state board of examiners composed of engineers. The educational examination and inquiry into the applicant's experience and reputation should be such as the examining board deems necessary to fit the applicant to perform the duties of the office. The educational examination should not be too technical. It is an honest, capable, practical, experienced man, rather than special technical ability that is needed under present conditions.

There are 101 county surveyors in the state, and about 10 county engineers, who are not county surveyors. This makes a total of 111 officials. Forty-five district highway engineers employed by the year so they could definitely plan their work, can, each with the assistance of one man about half of the time, do the work these 111 men are now doing, and in addition, all the other road and bridge work that will be required until Kansas begins to build the more expensive types of roads.

This means that about 65 competent men could take the place of these 111 officers. It also means that all the townships and counties would have the services of a capable highway engineer and county surveyor. Four counties do not have a county surveyor now and only 35 counties have county engineers and out of this number only about 20 counties give their county engineers a chance to do anything.

The district engineer should prepare plans and specifications, or check and approve them for the construction and maintenance of all the highway and bridge work in his district. He should make itemized estimates, check and approve bills, manage and supervise all construction and maintenance work, and the road dragging and all repairs. All contracts should be let, and all machinery and tools purchased on his recommendation. He should have authority to do work by force account, and to hire and discharge the men directly under him. He should appoint the township overseers with the approval of the township board. And he should be in every sense, an expert manager of highways and bridges and have full charge of the same in his district. All these powers, however, should be exercised subject to the direction and supervision of the county commissioners and general regulations of a state highway department.

To employ the kind of men needed to handle the work in each district, will require that they be paid a salary of from $1,200.00 to $2,000.00 per year, or an average of about $1,500.00 and their traveling expenses.

If the positions of county engineer and county surveyor were combined and a district engineer employed on a yearly salary, all survey fees should be turned over to the county road fund. It would, then, cost a total of about $100,000 per year or about 2 per cent of the state road and bridge funds to provide the services of competent engineers and surveyors for every township and county in the state.

Methods and Cost of Constructing a Brick-Cinder Road in Mississippi.

During the fall of 1912 the United States Office of Public Roads constructed an object-lesson road with a wearing surface composed of brickbats and cinders. The improvement forms a section of what is known as the Military Road, extending from Columbus, Miss., toward the Alabama State line. Work on the road was begun on Sept. 18, 1912, and was completed on Oct. 9, 1912. The following description of the work and cost data were obtained from Bulletin No. 53 issued by the U. S. Dept. of Agriculture:

Earthwork.—The land adjacent to the road is hilly and the natural soil is sand and clay. The length of the section improved is 2,128 ft. The road was graded 30 ft. wide in cuts and 20 ft. wide on fills, giving an area of 2,615 sq. yds. of roadway. The maximum grade of the road was reduced from 1.5 to 0.4 per cent. The earth was loosened with a plow, picks and shovels, hauled in wagons and slip scrapers, and spread with shovels. A total volume of 1,696 cu. yds. of earth were moved. The average haul was 250 ft. and the maximum 900 ft. The equipment consisted of a road plow, slip scrapers, and a small road grader. Labor cost $1 per day and teams $3. The total and unit costs of the earthwork are given in Table 1.

TABLE 1.—COST OF EARTHWORK FOR A BRICK-CINDER ROAD IN MISSISSIPPI.

Item—	Amount.	Unit cost per cu. yd.	Unit cost per sq. yd. roadway.
Clearing and grubbing .$	20.75	.....	$0.008
1,696 cu. yds. excavation	236.15	$0.14	0.090
Shoulders and ditches..	9.40	.....	0.004
Labor on pipe culvert	2.75	.....	0.001
General expenses .	2.90	.....	0.001
Total	$271.95		$0.104

Materials and Methods Used in Constructing the Wearing Surface.—The brickbats and cinders used for the wearing surface were donated by a local brick manufacturer. Both materials were loaded on dump board wagons at the brickyard, hauled an average distance of ½ mile, and spread upon the prepared subgrade with the shovels and potato hooks. The brickbats, which were used in the foundation course,

TABLE II.—COST OF BRICK-CINDER WEARING SURFACE FOR A ROAD IN MISSISSIPPI.

Item—	Amount.	Unit cost per cu. yd.	Unit cost per sq. yd. wearing surface.
Loading brickbats	$ 65.35	$0.202	$0.030
Hauling brickbats	133.25	0.413	0.060
Spreading brickbats	20.45	0.063	0.009
Loading cinders	63.70	0.087	0.029
Hauling cinders	110.65	0.323	0.050
Spreading cinders	12.10	0.035	0.006
General expenses	4.30	.....	0.002
Total	$409.80		$0.186

The following is a summary giving total costs:

Earth work	$271.95	.....	$0.104
Wearing surface	409.80	.....	0.186
Total	$681.75		$0.290

were broken with hammers at the road, and were spread to a width of 14 ft. and a depth of 4 ins. after compacting. The cinder surface course was spread to a width of 16 ft. and a depth of 4 ins. after compacting. The road was compacted principally by the action of traffic, but a short section was rolled with a road roller. The road was surfaced only on part of the section graded, a length of 1,243 ft. being covered. The area of the foundation course was 1,933 sq. yds., requiring 322 cu. yds. of brickbats. The area of the cinder course was 2,210 square yards as it was made 2 ft. wider than the foundation course, and 343 cu. yds. of cinders were used. The road is one of the most heavily traveled roads in the county, but judging from results obtained upon a similar road in the same locality, it is expected that the surface will wear well if it is given proper attention. The total and unit costs of constructing the wearing surface are given in Table II. It should be noted that material was donated, no rent was paid for roller, and no superintendence is charged.

Good Roads in the Philippines.—Two through highways are being built across the inland of Luzon, known as the Manila-North and the Manila-South roads. The former will be 350 miles long and extends from Manila to the town of Banguy in the northern part of the island. The other will be a traffic line from Manila to the southern extremity of Luzon, a distance of about 300 miles. There are something like 3,000 miles of roads in the islands, and they are maintained at an average cost of $700,000 per annum. New work is going on at the rate of 300 miles a year. The total amount of money expended for construction, maintenance and improvement of roads and bridges in 1912 was $2,193,023, of which $1,003,300 was insular government funds, $1,143,000 provincial government funds, and $47,324 municipal funds This amount does not include the expensive Benguet road.

Buildings

The Design of a Structural Steel Plant.

It is difficult for the designer to anticipate the requirements of a modern structural steel plant. In making a study of existing plants one is brought to a realization of the fact that most of these plants have started in a small way, and have developed as business has increased. In most cases no definite plan of expansion has been followed, and many plants after several years of operation find that their lay-out is not an economical one. The following article, which treats of the general principles involved in the design of a structural steel plant, has been abstracted from a paper by Mr. E. H. Darling, in Applied Science. This article, in a very practical manner, treats of the general principles of design, and illustrates these principles by reference to an ideal design. Conditions as they exist in Canada have been considered, but the discussion is applicable to the design of a structural plant in any part of this country not a structural steel center; while many of the principles are applicable to any kind of manufacturing plant.

A plant for the fabrication of structural steel for most markets must be equipped for a wide range of work. Even though greater efficiency can usually be obtained by specializing in one class of work it is not always possible to do so, as contracts of a uniform nature cannot always be obtained and almost any contract has a considerable variety of work in it. Certain classes of work, such as building and highway work, are usually not required during the winter months, while railway work can be done throughout the entire year. As we shall assume the plant to be a new one whose market is not assured, it should be laid out with the idea of doing general work and developed later along those lines which prove to be most profitable, as the company finds its place in the business of the country. To secure flexibility of capacity in a plant having an output of about 10,000 tons per year, it will be found advisable to have two main departments: First a structural shop for all kinds of light truss and beam work, with facilities for handling pieces weighing from five to ten tons; and second a girder shop where 50-ton girders can be handled economically. A plant constructed along these lines, with all the necessary subsidiary departments, will be able to fabricate any type of railway bridge up to 300-ft. span, and also can be operated efficiently on ordinary light structural work.

CONDITIONS GOVERNING THE LOCATION.

The location of a structural steel plant is subject to the same general principles which govern that of other manufacturing enterprises. Although there may be many secondary considerations which will have more or less weight, the four main ones are: (1) the market; (2) the supply of the right class labor; (3) accessibility to raw material; and (4) the supply of power.

The Market.—The market for structural steel depends on the class of work sought. Railway work is so widely distributed that good railway connections are of more importance, within reasonable limits, than location. Freight rates do not necessarily enter into the question, as the railways haul all such material and erection equipment, which is for the use of their own roads, free of charge. For mill building work the greatest demand will be in industrial centers, although much of the same class of work is used in large public buildings, such as armories, churches, auditoriums, rinks, etc. For beam and general building work, such as building contractors require, large commercial and business centers afford the best markets. As most of such material must be transported by team to the building site, good roads are necessary to handle this business. Steel highway bridges will be required to replace old spans in the well-settled rural districts, and new bridges must be built in rapidly growing sections.

In finding the "center of gravity" of a market, i. e., the point from which it will be possible to serve the largest number of customers, freight rates and cost of transportation rather than distances should be considered. For various reasons, due to questions of competition and peculiar conditions, it will often be found that it costs more to make a shipment to a nearby place than to one at a greater distance. The value of a market, however, depends largely upon the amount of competition which must be met; therefore in finding the center of gravity it will be necessary to assign "weights" to various cities or districts according to how well they are served by existing companies.

Labor Conditions.—The cost of labor in the manufacture of structural steel is such a large percentage of the total cost that the question of obtaining a sufficient supply of the right kind of labor deserves careful consideration. In the first place it will be necessary to secure experienced foremen and leaders, and these can only be obtained from other plants. With these as a nucleus it will be possible to train up intelligent laborers to do a great deal of the work, but, owing to the changes which are constantly taking place in a large organization, there will always be a need for men who have had some experience in a structural shop. Such workmen can more readily be obtained at some point near or within easy reach of other structural plants, while at other points it will be necessary to have a well-organized employment department to look after this. About two hundred men will be required to operate the plant, who, with their families, may mean that about one thousand persons will be supported by it.

Raw Material.—Apart from a small quantity of machine shop supplies, such as any manufacturing concern uses, "raw material" has only one meaning in the structural steel business, and that is the rolled steel plates, beams, angles, etc., used in fabricating. It is of great convenience to a structural steel company to be as near as possible to its source of supply. Not only will it mean a big saving in freight rates at times, but what is often of more importance, it will save delay in getting material, and when mistakes are made in shipments they are more easily remedied. The farther removed a plant is from its source of supply the more necessary it is that a large stock of material be carried. This requires considerable capital, but as a rule it will be found very profitable as well as a great convenience to carry such a stock.

Supply of Power.—Compared with many other lines of manufacture, a structural steel plant does not need as much power in proportion to its output, and for this plant not more than 250 HP. will be required. The most convenient form for it, as will be shown later, will be in electrical energy, and it is usually easy to obtain it in this form in manufacturing centers. But it may hapen that an otherwise desirable location does not offer this advantage; then a power plant must be installed. This means the investment of extra capital and an increase in operating expenses, which must be taken into consideration.

There are a great many secondary conditions which influence a choice of location, such as climate, special inducements from municipalities, personal preference, etc., all of which deserve careful consideration. It must be remembered that a mistake made at this stage will handicap the industry for all time.

CHOICE OF SITE.

An ideal site for a structural steel plant is not readily obtainable. A large area is essential, even for a comparatively small plant, while ample provision should be made for growth. Judgment must be used in this particular. Too much land might be a burden to a new company, or, on the other hand, might prove to be a valuable investment. The land should be level, and should have a water supply and good drainage. It must have at least one railway connection, and more if possible. For immediate neighbors it is convenient to have other iron industries, for these not only supply the smaller items of raw material, but tend to create a larger labor market. The site should be convenient to the homes of the working men, for, apart from the question of wages, no one thing has a greater influence on the problem of building up a permanent and efficient organization than this item.

It is unnecessary to discuss all the possible variations which may occur and still give a satisfactory site, for such items as the shape of the lot, the location and number of railway connections, position of streets, etc., make each site a special study. The necessary, or at least very desirable, features are:

(a) Sufficient room to arrange switches and buildings conveniently, leaving ample room for extensions.

(b) A storage yard at one end of the plant where cars may be unloaded and material stored.

(c) A corresponding area at the other end, smaller in extent, to serve as a storage yard for finished work, with facilities for loading it for shipment.

AN IDEAL LAYOUT.

Figure 2 shows an ideal arrangement for a structural shop on a lot 600 ft. wide and 1000 ft. long, containing about 14 acres. A railway from which switches can be made is assumed to run along the west side, and a public street along the north end. The main features of this plan are as follows:

The buildings are placed approximately near the center of the lot, leaving the ends clear for stock and shipping yards. The office, power house and template shop, which must be of a permanent type of construction, are placed along one side of the lot, where they will not interfere with future extensions of the plant. The storerooms and the smaller departments are arranged along the permanent side of the main building, for the same reason. The east side is left clear for additions.

The service tracks are placed between the buildings so that supplies and materials can be readily unloaded where required. It will be of great convenience to have two connections with the railway, as shown in Fig. 1. A double track permits the ready passage of cars, and also gives more storage room for them. A cross-over between the tracks will be of great service. One track should be run into the shop under the traveling cranes, so that long and heavy pieces can be loaded directly onto the cars. It is very necessary to have easy curves on this track as long girders, loaded on several cars, will be brought out over it. The curves shown have a maximum curvature of 20°, which should be considered the limit. At some future time another track may be placed along the west boundary, where cars of the erection department may be stored when not in use.

Regarding the arrangement of the various buildings and departments, it will be noted (see Fig. I) that the office is placed so as to be the first building approached on entering the plant. Space is left for adding a new front when larger and more elaborate offices are required. The power house, boiler room and template shop can conveniently be placed in one building, divided by fire walls, or they can be located in a series of buildings. The south end of the template shop may have a temporary end, which will allow it to be extended but the power house and boiler room should be made of large proportions, as it will not be convenient to enlarge these buildings.

The arrangement above described places the power house near one boundary line of the site so that high voltage power lines may be brought into it in such a way that all danger of accident from contact is eliminated. Further, it is located at a central point from which it is convenient to distribute the power to all parts of plant. A small tower at the north end of the power house forms the transformer room, and the main switchboard may be placed along the wall between this tower and the engine room.

As the plant will probably be run by electric power purchased from a power company and will not be generated at the site, the boiler equipment will be used largely for heating purposes. The buildings which require heat will be the office, power house, template shop and machine shop, and these are grouped around the boiler room so as to make the distribution system very simple.

LOCATION OF BUILDINGS

The product of the template shop is used entirely at the south end of the main shop, where the "markers" do the laying out. It will be desirable, therefore, to have the template shop as convenient to the markers as possible, as shown in Fig. I. It will also be within easy reach of the drawing office, which is over the business office.

On the other side of the tracks, convenient for shipping and receiving (and also at the permanent side of the plant), are placed the storerooms, machine shop, rivet and bolt shops, and the blacksmith shop. These departments are relatively small compared with the rest of the plant, and the space allotted should be sufficient for all requirements. Extensions, however, may be added at either end, and the partitions between these departments may be made of temporary construction so as to be easily rearranged should it be found desirable to make any changes in the future.

The position of the storeroom, with its platform along the tracks and its road from the street, permits suplies to be readily handled. Its proximity to the machine shop, rivet shop and blacksmith shops will save time in transporting materials to and from these departments. As far as the main shop is concerned, the largest items drawn from the storeroom will be rivets and bolts, and these are used entirely by the assemblers and riveters at the north end of the building. The storeroom is made of good size, as by far the larger part of the room will be taken up by bulky equipment for the erection department.

The main shop consists of two 75-ft. aisles, one for heavy girders and the other for general structural work. As the structural department may require an extension in the future the girder shop is placed adjacent to the storeroom, and the way is left open for adding two more aisles to the structural shop. Putting the girder shop in this position makes it possible to bring the railroad track into it.

TYPE OF CONSTRUCTION FOR BUILDINGS.

The office building, power house and template shop should be substantially constructed so as to be warm, dry and fireproof. The office will be two stories high, the second story being occupied by the engineering staff. The power house and template shop will be a one-story brick building with a monitor. This building has a 50-ft. span and a 16-ft. clearance, the roof trusses being of the Fink type. The roof is of 2-in. concrete, carried on steel purlins and trusses. The windows, which take up a large part of the wall space, will have steel sash. The monitor windows will be on hinges so that they can be opened, and sections of the other windows will be made to open. The floor of the power house will be concrete. The template shop floor will be raised about 18 ins. above the ground level and will be of 4-in. mill construction with an upper floor of 1-in. white pine, which makes a good floor for laying out work.

The type of construction to be used in the main building is shown in Fig. 2. A large percentage of the exterior walls as well as the monitors will consist of windows which will insure good illumination throughout. The remainder of the wall area will be corrugated iron, with the exception of the 8 ft. below the first row of windows. This will be made a 9-in brick wall on the north, west and south sides of the building, with a concrete window sill to give it a finished appearance. This brick wall will make the building much more comfortable both in winter and in summer.

For the roofs of all buildings which are subject to considerable deflection as well as vibration and shock, it is preferable to use 2-in. matched wood sheathing rather than concrete. As the roof is flat the covering may be 4-ply felt with tar and gravel.

The framework will be steel throughout and will be entirely self-supporting. Steel sash will be used for the windows, as it is only by its use that the large windows can be secured.

The main building is 200 ft. wide and 500 ft. long. It consists of one 50-ft. and two 75-ft. aisles with a 60-ft. transverse aisle at each end. An economical length of bay will be 20 ft. The clearance between the bottom chord of the truss and the floor in the 75-ft. aisles should be 21 ft. to allow for air hoists, etc. In the transverse aisles and in the girder shop additional headroom must be provided to allow for traveling cranes. By making the roof at these points as high as the roof on the monitors a good appearance will be obtained. In the machine shop aisle, 15 ft. will be sufficient clearance. Too much height here means a more difficult building to heat, and longer belts for belt-driven machinery.

All trusses whose bottom chords are at the 15-ft. and 21-ft. levels should be designed to carry two 5-ton trolleys. A factor of safety of five should be used for one trolley and of three for two trolleys. For the roof live load at least 50 lbs. per square foot should be provided for and, for certain localities where the snow fall is unusually heavy, the live load should be increased. Where there is any possibility that a column will have to carry a jib crane, provision should be made for it in the original design. This refers to the columns in the girder shop where those along the west wall will have to take radial reamers, while 5-ton traveling jib cranes for riveting machines have to be supported at the east side. Particular care should be taken with the design of the bracing for the building, and the bottom chord system should be especially heavy to tie the whole structure together rigidly.

To facilitate the transfer of material from one aisle to another, the interior columns in the structural shop should be so arranged that the bottom chords of the trusses form one continuous trolley runway from one aisle to another. This is done by spacing the columns 40 ft. on centers, placing them in the middle of a bay, and carrying the roof trusses on longitudinal trusses between these columns. The east wall is filled with temporary columns to carry the girts and siding. These columns can be moved further east when another aisle is added. It will be necessary in order to attain economy in shop work to floor the main building. All things being considered, it will be found that a wood floor will be comfortable for the workmen, reasonably durable, easily repaired, and as cheap as any in first cost. On account of the heavy loads which the floor must sustain and the rough usage it will receive it should be made of 3-in. pine. Nailing pieces or stringers of 4x4-in. timbers should be imbedded in cinders, thoroughly rammed and smoothed flush. These pieces should be spaced from 3 to 4 ft. apart. Every precaution should be taken to keep the sub-floor perfectly dry, and if there is any possibility of the wood getting damp, it should be treated with some kind of preservative before laying.

MACHINERY EQUIPMENT.

The machinery equipment of the plant is probably the most important question to be considered. Not only does the output of the plant depend on it, but a large proportion of the capital of the company may be invested in equipment and, unless every department is nicely balanced, more or less of it may be unproductive much of the time.

The manufacture of structural steelwork requires in its operations the straightening, cutting, shearing, punching, assembling, reaming, riveting, milling and general finishing of a large number of different steel sections. It would, therefore, be necessary to provide at least machinery for these operations. But the capital invested in a manufacturing plant is subscribed to turn out a certain amount of work which will net a certain percentage of profit. It will therefore be necessary to have enough machines of each kind to make the capacity of the plant what is desired. For reasons mentioned before, the proportion of each class of work varies widely with different contracts. It therefore becomes a matter of experience and judgement to fix on the amount of equipment to install, as the most careful figuring is at best very approximate. The first item which it is well to fix is the number of riveting machines required. From this the number of punches may be decided upon, and the rest of the equipment proportioned accordingly.

In purchasing machinery certain general rules should be observed. It will be found advantageous in the long run to buy machines having a rated capacity well above the average required. Besides the fact that the heavier machine will be more durable, it will in some cases give the plant an equipment for a wider range of work at comparatively little extra cost. At the very least it will provide for an emergency which otherwise might prove very expensive.

If the demand for a certain machine product varies widely it will sometimes be found economical (when the cost of operating a large machine is high) to have two smaller machines instead of a single large one, the first with sufficient capacity for the average requirements and the second capable of taking care of the extraordinary demands. As the plant spreads out, duplicate machines may be required at different locations, it being cheaper to operate two machines than to transport the material long distances.

While complicated machines are to be avoided it will save capital at the start to buy machines which may be changed for different classes of work, provided they are not in constant use. For example, the plate rolls should be made a combination of bending and straightening types. The punches should be made so that either cross-cutting or splitting shear blades may be put on to replace the punch and die. The light riveting machine should have one or more extra interchangeable stakes to be used for column or other special work. In this way, at the cost of the extra parts and the time it takes to make the change, work can be done which otherwise would require two separate machines, one of which would probably be idle most of the time.

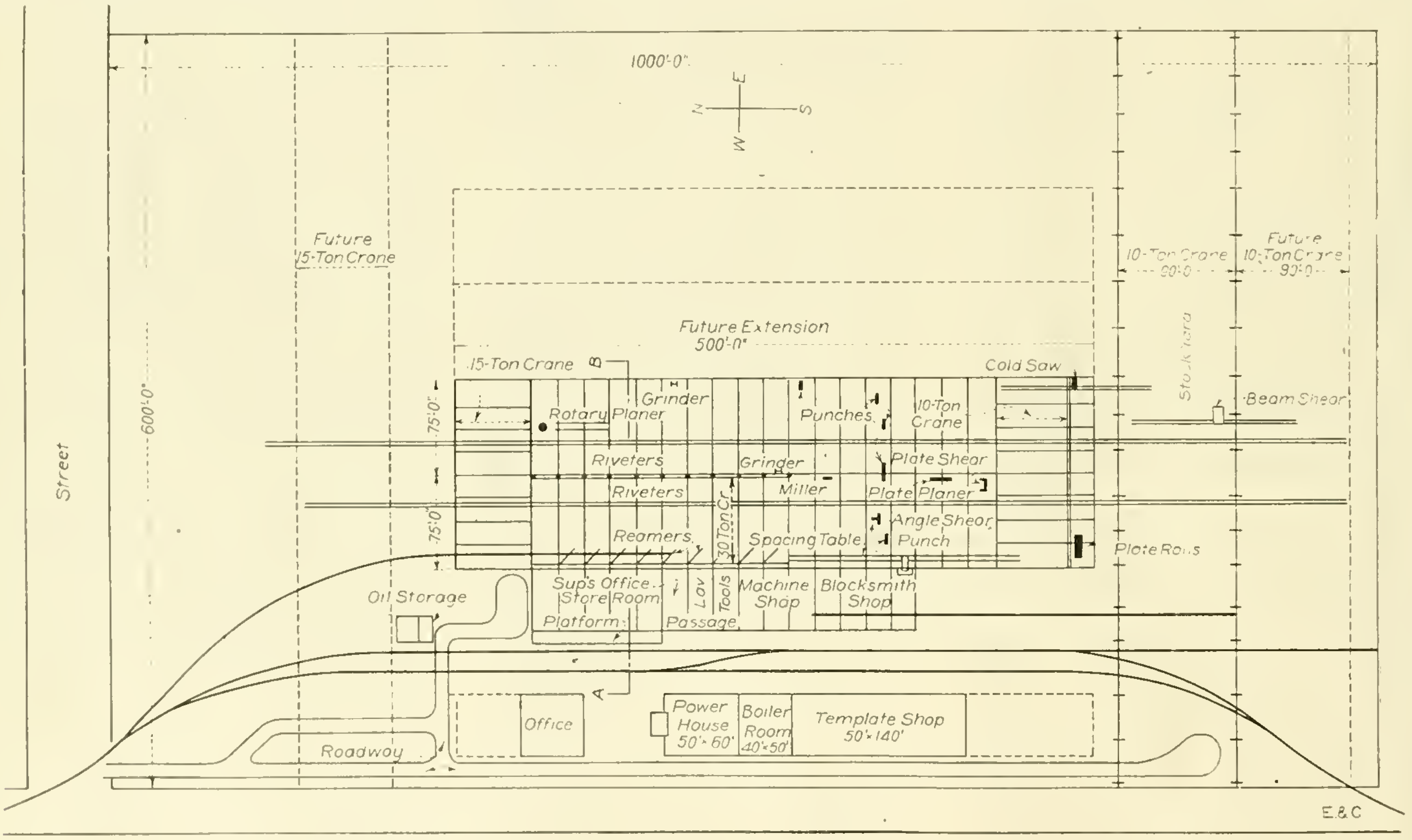


Fig. 1. Plan of Ideal Lay-Out for Structural Steel Plant With About 10,000 Ton Capacity.

The matter of punches, shears, reamers, riveting machines and other machines might well be taken up at this stage. Their capacities, accessories and other features should have careful attention, but the discussion of these items is outside of the province of this preliminary investigation of the general lay-out.

ARRANGEMENTS OF MACHINERY AND ROUTES FOR MATERIAL.

The arrangement of the machinery and the establishment of the routes for the material to pass through the shop is one of the most difficult and perplexing problems in connection with the design of a structural steel plant. In theory the material should enter one end of the shop and pass through it from one department to another, emerging at the other end as finished product, without having once been turned around or carried back over any part of its route. In practice, however, there is such a variation in the order in which the work must be performed—due to details of design and special requirements—that no fixed route can be maintained. At best a material route can be laid out only in a general way, observing the following principles:

Arrange the plant for that class of work which promises to be the most plentiful. If it is necessary that some material be rehandled, let it be the shorter and lighter pieces rather than the long heavy ones.

Sufficient space, 65 to 70 ft. in the longitudinal direction of the shop, should be left between machines so that, under ordinary circumstances, there will be no crowding or interference. Material over 70 ft. in length may be considered very unusual and when it becomes necessary to take care of it, more or less difficulty and interference may be permitted. To design the whole plant for it would add very materially to the cost of buildings, while the increase in the distances that the shorter material would have to be transferred would add to the cost of operation of the plant on regular work.

The material will be sorted out in the stock yard, and any piece with kinks or sharp bends in it will be taken directly to the straightener or plate rolls, as the case may be. It will then be put on the trucks, taken into the shop and piled on the markers' skids. From here it will be taken to the various cutting machines, and then on to the punches. As it is punched it will be stacked up until required for assembling, when the various pieces are bolted tightly together, taken to the riveters and riveted up. Reamed work will, of course, sometimes be required. The work may now be finished up, cleaned and painted; and as soon as the paint is dry it is ready to ship.

A study of Fig. 1 will show that the machines and departments are arranged approximately in the order mentioned above, starting at the south end of the plant. Variations from this rule are due to physical limitations, or are the result of experience.

HANDLING MACHINERY AND INDUSTRIAL TRACKS.

Next to the selection of the machinery there is no one feature about a structural steel plant which should receive more careful consideration than the handling equipment. The cost of handling is the largest item in the labor account, and the capacity of the plant depends a great deal on the speed with which material is fed to and taken away from the various machines. Even after every unnecessary motion is eliminated, there is considerable handling of each piece, and the weight, length and flexibility of most of them require care and special appliances.

In the ordinary run of work it is seldom that a piece of material from the steel mills will exceed three tons in weight. The stock yard crane, however, should have at least ten tons capacity, as it is usually desirable to lift several pieces at one draft. Only one crane, with a 90-ft. span, traveling the full width of the lot, will be needed at first. At some future time another crane and runway may be added, which will not only increase the handling capacity, but will double the area of the stock yard.

This yard crane will be used to unload material from the cars and to sort it. As the material is required in the shop, it will be put on small trucks which run on narrow-gage tracks from one end of the shop to the other. Inside the building the hoists on the bottom chords of the trusses afford a means by which the material may be taken off the trucks and transferred to any point across the shop. At each end of the building, however, on account of the amount of handling which is required at these points, it is advisable to have traveling cranes of about 60-ft. span—a 10-ton crane at the stock yard end and a 15-ton crane at the shipping end. These cranes will also be found of great service in transferring material from one aisle to another.

At the punches longitudinal trolley beams with trolleys and chain hoists are necessary to hold the work as it is fed through the machine. These trolley beams must be suspended below the trusses and have openings through which the trolleys on the trusses can pass. Special travelers and jib cranes can be arranged to suit the need of any particular machine.

In that part of the girder shop where the girders are assembled and handled, it will be necessary to have two traveling cranes, each having a 30-ton capacity. Traveling jib cranes for carrying riveting machines can be arranged to run along the east side beneath the overhead cranes. In the structural shop the riveters will have three-ton capacity traveling cranes with 15-ft. span, and will be carried on runways suspended from the roof trusses.

Three-ton and five-ton air hosts will be required wherever much lifting is to be done, provided the headroom is sufficient for the hoist. For holding the work at the machines, however, a chain block is better, as air hoists are not steady. Wherever possible the material will be laid on horses or skids, so as to decrease the raising and lowering of it. If much movement is required at any point rapid-acting blocks should be used.

It will be found convenient later to have a 30-ton traveling crane with a 75-ft span in the yard at the north end of the plant. Under it finished material may be stored and loaded for shipment as required. This crane can be used for assembling large trusses when it is necessary to put them together at the works before shipment.

All of the 30-ton cranes should have five-ton auxiliary hoists, or should be provided with changeable gears, as some cranes are now made. This will save a great deal of time when the crane is used for handling light pieces, the slow motion or main hoist being used only for heavy loads.

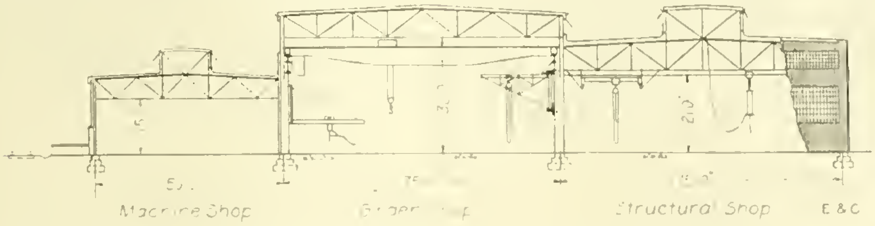


Fig. 2. Cross-Section of Main Building of Structural Steel Plant, Showing General Dimensions and Type of Cranes and Hoists.

The handling equipment under certain conditions should include a ten-ton locomotive crane. Such a machine will be found of great service during construction for unloading and placing building materials, machinery, etc. It will also be a great convenience, if not a necessity, in moving cars, unless unusually good shunting service is available from the railway company. It may be used to good advantage by the erection department for certain work.

All parts of the handling equipment should have a wide margin of strength. Chains, hooks, cables, chain blocks, etc., which are frequently overloaded, soon give trouble and become dangerous. The most serious accidents in a structural steel shop are the result of falling material.

POWER AND LIGHTING SYSTEMS

Electric power is usually delivered as two or three-phase alternating current at a high voltage to be transformed to suit the requirements of the purchaser. For distribution about the plant a voltage of 220 or 500 volts is usually used. The higher voltage requires less copper in the feeders, but for a structural shop the lower voltage is preferable. The presence of so much metal and the rough treatment which wires and conduits are liable to receive make it unwise to use a voltage which might be dangerous to workmen.

The power equipment must include a motor generator set, as direct current will be required, for a variable-speed motors such as cranes, reamers, etc. The motor side of this set should be of the synchronous type with sufficient capacity to correct the power factor of the plant. On account of the varying loads on the motors the power factor is sure to be very low. The capacity of the generator should easily be able to take care of the needs of the plant, including the lighting system and the extra traveling cranes which might be added in a year or two. A set with a capacity of 100 kilowatts direct current and a synchronous motor of 125 k.v.a. should give satisfaction.

The high voltage current will be brought down the tower at the end of the power house to the transformers, and from thence the low voltage current will be taken in conduits to the bus-bars of the switchboards. A separate switchboard will be needed for the direct current system. From the switchboards feeders in conduit will run to the machines in the power house and template shops. For the main shop the feeders will pass up the tower and overhead to cross-arms on the trusses of the building. A separate system, each on its own switch, should be provided for the different departments, so that any one department can be cut out without interrupting the other. There should be three sets of alternating current feeders, one for each aisle of the shop, with space on the cross-arms for the additional aisles to be added later. There should be two pairs of direct current feeders, one for the north cranes and one for the south cranes. If separate switches are provided at convenient points for cutting out each crane, it will hardly be necessary to have separate feeders for the other direct current machines in the shop. There will only be one or two of these machines in addition to the reamers.

All wires above the bottom chord of the trusses will be carried on cross-arms bolted to the steelwork, but wires below this level must be put in metal conduits. The starting switches for each machine should be as near the machine as possible. For the portable reamers a series of outlets must be provided on the columns about every 40 ft. down the shop, so that the reamers can be connected up by means of a plug and a flexible cord wherever needed. Each outlet and in fact every branch circuit must have a separate cut-out. In this and in all other particulars the rules of the fire underwriters must be followed, and every precaution must be taken to prevent short circuits and injury to the workmen.

For general illumination modern direct current flaming arc 110-volt lights give a suitable and an economical light. One light in every other bay, arranged alternately on the right and left-hand side of the center of each aisle, will be sufficient for general illumination, while they may be spaced closer over the laying-out skids and the assemblers. As the power circuit is 220 volts, it will be necessary to connect two lights in series. Each pair must have a cut-out and every four lights should be controlled by a two pole switch.

Sixteen candle power incandescent lights should be provided for each machine. These must be on good lamp chord and protected by wire cages having wooden handles. The power for these lights may be obtained by tapping the circuit which supplies the machines.

For the template shop the best illumination can be obtained by means of 100 watt tungsten lights suspended from the roof, three in each 20-ft bay. Along the walls, over tables, or near machines, 16 candle power carbon lights on drop cords will be used to give special illumination when needed.

COMPRESSED AIR SYSTEM

The transmission of power by means of compressed air is a very expensive method but it is so convenient to handle, and can be adapted to so many uses, that it would be impossible to get along without it. A pressure of 90 to 100 lbs. per square inch is generally used for operating pneumatic riveters, hammers, reamers and hoists. The plant can easily be supplied by a 700-cu. ft.-per-minute, two-stage compressor. When the demand gets beyond this it will be advisable to add a 1,400-cu. ft. machine to the equipment, and the piping, etc., should be installed at the outset with this object in view. By having two machines of different capacity the varying demand for air can be supplied more economically than by one large compressor.

The compressor will be belt-driven by a 125-HP. induction motor. It should have an automatic throttling device which will cut off the supply of free air completely when the pressure in the system reaches its maximum. Such a governor can be adjusted to regulate the pressure within 3 lbs. This small variation need not be considered, as the method is much more economical in power than if a regulator were used which would permit just enough air in the intake to maintain a constant pressure. It causes great fluctuations in the power used, however, as the compressor runs light when the intake is cut off.

In order to keep the pressure in the distribution system as constant as possible and to relieve it of shocks it is necessary to have a storage tank near the compressor. This tank will also act as a cooler, and in it will collect a great deal of the moisture which would otherwise pass into the distribution pipes. A large second-hand fire-tube boiler can be converted into a splendid storage tank. It should be set in a vertical position so as to allow a natural circulation of air through the tubes. The inlet should be near the top and the outlet should be about 18 ins. from the bottom. A convenient place for this tank is in a corner of the boiler room. From it the air main can be taken underground to the center of the main shop and from thence by three vertical branches to the roof trusses, one branch for each of the air hoists in the girder shop and structural aisles and one branch with drop pipes down every other column of the center row, with outlets for the riveting machines, reamers, etc.

All horizontal pipes should be given a slight downward slope in the direction of the flow of air so that any moisture which is carried past the storage tank will not obstruct the flow. At the tank and at all low points in the system blow-off valves must be provided to the drains, and these should be opened at regular intervals. Care in this particular and abundantly large pipes will do away with all trouble from frost in winter. It will not be economical to provide special means to re-heat the air before using, and it is not convenient to arrange a satisfactory indirect method.

A low pressure system—about 20 lbs. per square inch—will be required for the oil forges. The blower, which will be driven by a small motor, can be supported between the trusses. The distribution system, which will be made of galvanized iron pipes, will be carried overhead and down the columns near the rivet forges. Ultimately a separate blower will be required for the rivet-making plant and the blacksmith shop, but for the present the one machine will suffice.

WATER SUPPLY SYSTEM

Apart from the quantity of water required for drinking, sanitary purposes and fire protection, it will only be used for the water-cooler of the compressor and in the boiler of the heating plant. A 6-in. pipe will supply enough water for several fire streams if the pressure is good. The source of supply will probably be a public main on the street to the north of the property. The best place to bring in the private main is along the west boundary. Here it will be always accessible and not likely to be covered up with materials, etc., and furthermore, in case of a break it cannot do much damage to tracks or foundations. Where the first branch in the main occurs a manhole should be built, and a valve should be placed on both lines leading away from it. In fact it will be found of great convenience to have plenty of valves in the system so that any one section can be cut out with the least possible interference with the rest of the system. It is most important that the supply for the water-cooler on the air compressor be as independent as possible, as the lack of water here will practically stop the operation of the plant.

Hydrants should be placed at convenient points, but not too near the buildings. By having them near the tracks they may be used for supplying the locomotive crane. An automatic sprinkler system should be installed in the template shop.

The ends of all branches should be arranged with plugs or caps to permit an extension as the plant grows, and tees should be put in at points wherever there is the slightest possibility that a branch might be needed in the future.

SEWERS AND DRAINS.

The drainage system and its arrangement will depend entirely upon local conditions and upon the amount of fall obtainable. With a minimum grade of ⅛ in. in 1 ft. for a 12-in. glazed tile sewer it will require three such lines to properly drain the plant as first constructed. Assuming that they empty into a trunk sewer on the street and that they are to be 4 ft. deep at the manhole in the stock yard, the trunk sewer should be at least 13 ft. deep where the drains enter into it. If the depth is a little more than this, a saving in cost might be made by running one 24-in. sewer about 300 ft. into the property and connecting the 12-in. sewers to it. The question of sewers, however, depends so entirely on local conditions that nothing in the way of a definite design can be worked out at this stage. In general, however, it may be said that in addition to providing for carrying off the rain water from the buildings and the sewage, the surface water which may collect on the site must be taken care of. Special care should be taken to keep the sub-grade of the tracks well drained. All pits for machinery and other low spots should be connected with the sewers as well as all drain-offs and drips from water taps, steam pipes, etc. Sewers other than tile drains should not run near foundations if it can be avoided. In case such arrangement is found necessary the foundations must be carried down below the level of the sewer. Manholes with catch basins should be built at all junctions of the large sewers.

FUEL OIL SYSTEM.

Where light fuel oil of a uniform grade is readily obtainable at a reasonable price its use in rivet-heating forges and other furnaces is well worth considering. The rapidity with which a furnace can be heated, the ease in obtaining a uniform temperature, and the absence of the dirt incidental to the use of coal and coke are all points in its favor. Satisfactory results can only be obtained from a correctly designed system, and while for a small plant the first installation will be expensive it may be easily extended in the future at very little cost.

The great secret of success with fuel oil is clean oil of a uniform grade and large distribution pipes. For storage purposes two 12,000-gal. iron tanks will be required. These are to be buried in blue clay near the tracks at least 30 ft. from the other buildings, so that a tank car can be unloaded into them by gravity. The tanks should have one end exposed in a pit or concrete cellar, roofed into protect it from the weather. This pit should be large enough to contain a small electric pump which will be required to maintain a sufficient pressure in the distribution system. The first tank serves as a receiving and settling tank, and the second one is connected with the first by a pipe so as to draw off the oil a foot or two from the bottom of the first tank. It will be necessary to drain the sediment from the settling tank at regular intervals.

The distribution system should not be made of less than 1¼-in. pipe, and it should form a closed circuit or duplicate system, with a return pipe or overflow back to the second tank. By this means the oil is kept circulating at all times, and congestion from any cause is prevented. If trouble should occur at any point that section may be cut out by means of valves, and the rest of the system need not be interrupted.

For keeping the oil from congealing in winter steam coils should be put in the storage tanks, and the distribution pipes should be laid in an iron casing through which steam may be blown.

HEATING SYSTEM.

The problem of heating structural steel plants deserves special consideration in each case. It is a problem which is involved in the choice of a location. To heat brick buildings of ordinary type is not necessarily a difficult problem. The office can be heated by means of steam coils in the usual way. For the template shop, compressor room, and machine shop a hot air system with fan will give satisfactory results if properly designed. This system is cheaper to install than steam coils and interferes less with the arrangement of the benches, machines, etc.

The heating of the main building is a more difficult problem. The volume of air to be heated is so great (uncontrolled by partitions), the conductivity of the corrugated iron and glass which forms the walls is so high, and the large doors have to be opened so frequently that the cost of a plant which will give satisfactory results is almost prohibitory, to say nothing of the cost of operating it. Then, again, the capital invested in a heating plant lies idle a large part of the year. A heating plant is often unnecessary, as "salamanders" or open fires which burn coke or charcoal can be located at convenient points. The cost of such fires, even considering the time lost by workmen in keeping warm, will not equal the interest on a heating plant and the cost of operation.

If care is taken to make a corrugated iron building airtight it will be found to be much warmer than such a building is usually expected to be. Large glass areas, such as are becoming so common in manufacturing buildings, if in a south wall will produce the same warming effect which takes place in a greenhouse. This will add very materially to the warmth of a building on bright days, but it must be counteracted in summer time by good ventilation and circulation of air. Ribbed glass should be used to diffuse the light.

ERECTION EQUIPMENT.

The equipment required for erecting steelwork is merely mentioned here to call attention to it as being an item not to be overlooked. Such equipment depreciates so rapidly and the requirements vary so much with every contract that a large part of the cost of it should be charged to the work on which it is first used. This should be taken into consideration in making up the estimate for a piece of work. Special equipment, such as travelers, derrick cars, etc., can safely be left for consideration until the time comes to take contracts which require their use.

Water Works

Some Features of the Design and Construction of the 12,000,000-Gal. Mechanical Water Filtration Plant at Evanston, Ill.

The open end of the water supply intake of Evanston, Ill., lies only 1 1/16 miles from the shore line of Lake Michigan and is well within the range of drift for the sewage of Evanston and Wilmette, the greater portion of which is discharged on the lake front. In the winter of 1911-1912 a severe threatened epidemic of typhoid fever was averted only by the emergency use of chloride of lime. Filtration was agitated in 1912 and in August of that year Mr. Langdon Pearse and Mr. W. W. Jackson made a study of the conditions and submitted a report recommending the construction of a mechanical filtration plant. Following the report a bond issue was authorized by popular vote and plans and specifications were prepared, by Messrs. Geo. W. Fuller and Langdon Pearse, to govern the construction. The information here given relative to the design and construction of the plant is taken from a discussion by Mr. Pearse before the Western Society of Engineers.

DESIGN.

The plant is designed for a capacity of 12,000,000 gals. per day to serve an estimated population of 30,000. The reason such a large capacity was provided for is that the daily average pumpage in the summer frequently runs 12,000,000 gals. per day for a continued period of hot weather, with peak loads for four hours as high as 17,000,000 gals. per day in the afternoon. The average daily pumpage by the year has been about 6,000,000 gals. per day for the last ten years, the effect of increase of population having been taken care of by inspection of services and leaks. This large use of water is due principally to lawn sprinkling, since the pumpage drops suddenly in summer time on cloudy, and, particularly, on rainy days. With the condition in mind of continued daily loads, and the possibility of a healthy growth of population, 12,000,000 gals. capacity seemed a nominal size for this community. This rating is based on the usual rating of 125,000,000 gals. per acre per day.

The filter plant is located on Sheridan Road about one mile north of the center of the city on a site adjoining the Northwestern University and 300 ft. from the existing city pumping station. A general plan of the tanks, filters and mechanical equipment of the plant is shown in Fig. 1. A low-lift pumping station is to be built adjoining the present pumping station, which will be equipped with three centrifugal pumps, two of 10,000,000 and one of 6,000,000 gals. per day capacity. These will be geared to steam turbines, the entire outfit being furnished by the DeLaval Steam Turbine Company, with a guaranteed duty in excess of 69,000,000 ft.-lbs. per 1,000 lbs. of dry steam for the larger units. A 30-in. cast iron force main will conduct the raw water to the entrance to the mixing chamber, a Venturi meter being placed on the line to measure the pumpage.

The mixing chamber, shown in Fig. 2, is about 96½ ft. by 14 ft. in size, with a depth of 18 ft., having a total capacity of approximately 170,000 gals. giving a nominal mixing period of 20 minutes. The basin is to be constructed entirely of reinforced concrete, divided into bays by wooden baffles to maintain velocities of approximately 1 ft. per second. The mixing chamber lies between and is part of two coagulating basins, each about 39½ by 96½ ft. in size, and 15 ft. deep, having a combined capacity of approximately 830,000 gals., giving a nominal period of sedimentation of 1 hour and 40 minutes. These are to be constructed entirely of reinforced concrete with a groined arch roof, each compartment being divided by two concrete baffles into three bays to insure circulation.

The coagulated water will flow in a concrete channel across the space left for the extension of the filters to six rapid-filter units, each 23½ by 36 ft. inside dimensions, having an aggregate filter surface of 4,425 sq. ft. The filters will be grouped on both sides of a pipe gallery, over which is a platform with an operating table for each filter. All the filter tanks, floors and roof will be of reinforced concrete. Hydraulic valves will be used throughout, the controllers for the filters being of a modified Vivian type. The wash-water drain, filtered-water conduit and raw-water conduit will be built one over the other, forming a reinforced concrete structure down the center of the pipe gallery. The house covering the operating floor and a part of the filters will be of brick with a concrete roof, the inside being lined with a light yellow vitrified brick. The design is so made that the operator can see the entire filter bed at time of washing.

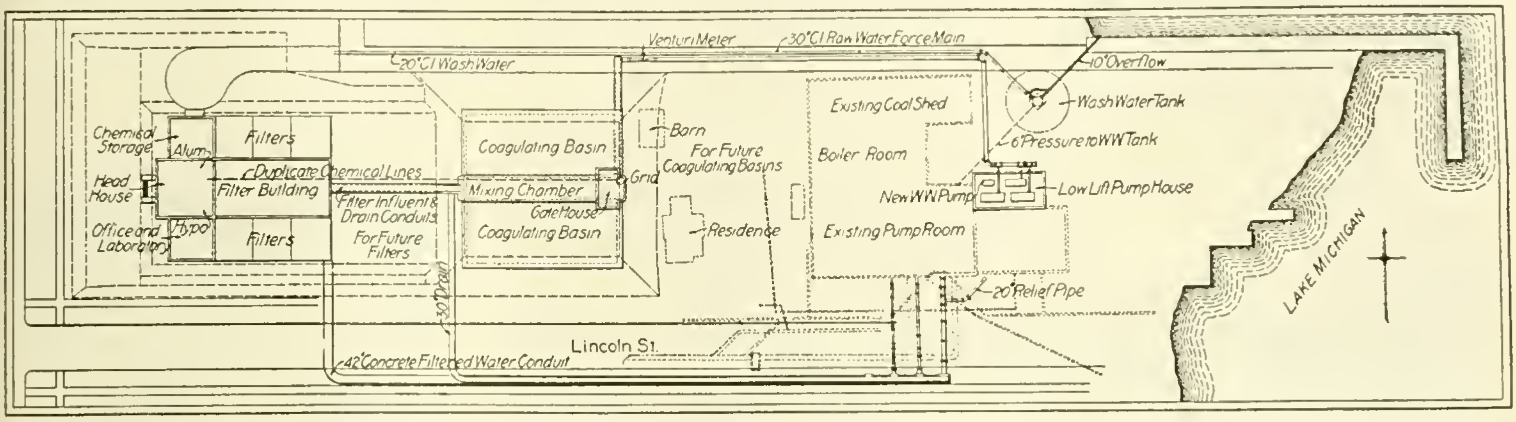


Fig. 1. General Plan Showing Relative Positions of Tanks, Filter and Mechanical Equipment of Evanston's Filter Plant.

Both filters and head house are placed above a covered filtered water reservoir, 154 by 161 ft. in plan, 12 ft. deep, divided into two basins, having a total capacity of approximately 2,000,000 gals. This basin has a groined arch roof, reinforced where necessary to carry the weight of the head house and filters. The standard spacing for the arches is 12 ft. 6 ins. on centers, with a rise of 2 ft. 9 ins. and a thickness of 6 ins. at the crown. The foundation is of firm clay so that the thickness of the floor arches is only 6 ins. at the thinnest point.

The two-story head house at the head of the filter gallery has two wings, one story each, built of reinforced concrete and brick with a tar and gravel roof. The dimensions of the head house are approximately 28 by 91 ft., and of the filter wing 75 by 36 ft. In the head house will be placed the tanks and apparatus for the proportioning and application of the coagulating and sterilizing chemicals, the recording gauges, laboratory office, and chemical storage. The second floor of the head house will be served by a hydraulic elevator.

The design of the rapid-filter units includes an under-drain system with perforated brass plates, set at the bottom of valleys in the fashion of the Minneapolis, Grand Rapids, and other plants. A novel feature, however, will be the omission of the wire screen, and the use of 12 ins. of graded gravel, on top of which the sand will be placed. The wash is to be high-rate water wash, supplied by a gravity feed from a steel tower tank of 100,000 gals. capacity to be erected behind the present pumping station on the lake front by the Chicago Bridge and Iron Works. It will be fed by a bleeder from the present high service mains. Fluctuations in the pressure due to the drop of level in the wash-water tank will be taken care of by a controller to be placed in the line at the entrance to the filter gallery.

CONSTRUCTION METHODS.

The work is being carried on under a general contract with the Norwood Engineering Company, Florence, Mass., the concrete work, head house, and excavation being sub-let to the Macdonald Engineering Company, Mr. James Macdonald, President. Owing to the situation, it was impossible to bring material in other than by team, a haul of approximately one mile being required from the nearest railroad switch. The contractor, therefore, has developed a plant for the elevating and storage of the gravel and sand in bins to minimize the handling by hand. The teams dump into a hopper from which a belt conveyor delivers the material to the sand or gravel bins. Storage sheds for cement are built alongside the bins. A Ransome concrete mixer discharges into two-wheeled buggies on an elevator for raising or lowering the cars of concrete to the appropriate level for placing. One inch yellow pine flooring has been used for form work wherever possible, with 2 by 4 in. studs spaced about 16 ins. on centers tied with very heavy wire.

The inverted groined-arch column bases were formed up continuously in rows by the use of a "spider," a metal frame consisting of 1-in. angles making a square, from each corner of which malleable castings of the shape of the groin converged to the base at the center, where they were held together by plates. In use, the angle frame was set on the two 6-in floor form strips and held there by pins. After running a cut-off strip over each pair of legs and getting the base shaped, the form was removed to the adjoining base and the corners smoothed up.

The groined-arch forms for the roof have been built in a manner different from that usually employed on such work. The form units are framed in the carpenter shop and transported to place on a pair of wheels. They fill the squares between the column corners, leaving a width approximately that of the column to be filled on the job, with short pieces of 1 in. flooring nailed in place by carpenters. This method was feasible because of the short span. The groined-arch forms were carried on stringers resting on the verticle tie studs of the column forms. The forms have been stripped at a minimum time of five to seven days, ten days being secured wherever possible. For the construction of the 30 in. wash-water drain and 42 in. filtered-water conduit, suitable circular collapsible forms were built of wood. At the bends a built-up form has been used, made of laths, plastered smooth with ivory wood-fiber plaster, smoothed up with paraffine wax dissolved in kerosene. The forms were destroyed before removal.

The excavation has been made on all the mass work with a Thew shovel, loading two-horse bottom-dump wagons with which the spoil was carried to adjoining lots or dumped on the lake front. The earth was fairly stiff, so that no sheeting was required except in trenches left open for days. The upper 5 ft. of excavation was loam and sand containing some water, under which a clay stratum was found, free from water. Some of the sand from the excavation was suitable for use in the concrete, but the major portion of the concrete was made with torpedo sand and gravel brought from pits along the Fox River. The imported sand was found somewhat unsuitable, owing to the lack of fine material, so that a mixture was made of one part plastering sand and five parts torpedo sand for general use. This made a nicely-working tight concrete.

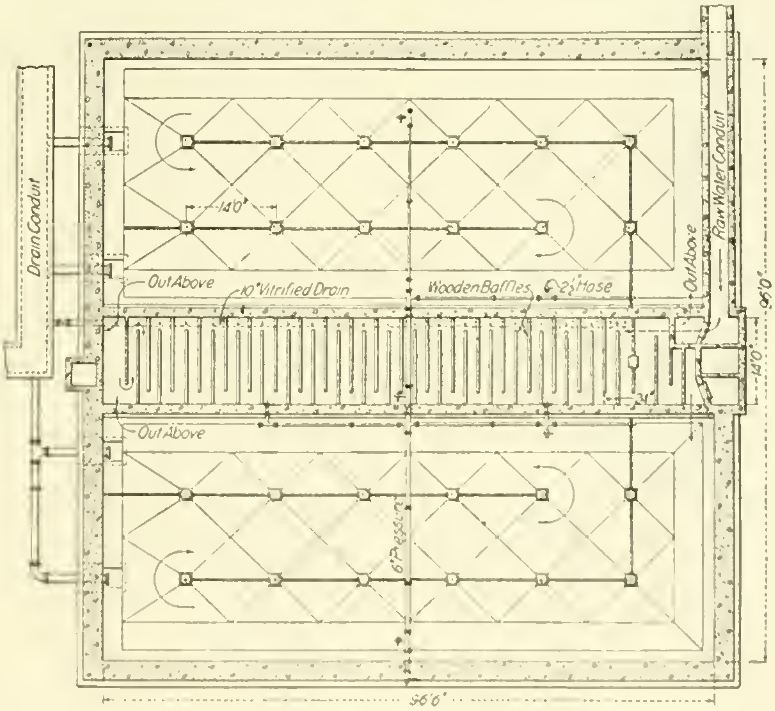


Fig. 2. Plan of Mixing Chamber and Coagulating Basins, Showing Baffles, Evanston Filtration Plant.

The work has progressed through the completion of the filtered water basin. Two of the filters and the head house are in process of erection, and the coagulating basin is well under way. It is hoped to have the work completed by the contract time, May 31, 1914.

A new method for forming concrete was described by Mr. James Macdonald, following the discussion by Mr. Pearse, as follows: We are using a new method for forming concrete, under certain conditions. The scheme is a little different from the ordinary construction, and is what we call a moving form. It consists of a wooden mould made to represent the work that is to be built. The walls are inclosed in a wooden frame structure with which it is spanned, and a suitable jack mechanism, having a U-shaped yoke, is provided, which spaces the faces of the walls that are filled with concrete. The jack is supported by 1⅛ in. vertical rods from the foundation, and these rods are continuous from the top to the bottom. The whole mechanism is carried by the jacks so that the concrete supports nothing but itself during construction. The weight of the form, the steel work which is going into the form, also of the men and material, is not carried on the concrete which we are moulding. In starting, we deposit a small amount of concrete in the form, and this is allowed to stand until it is partly set. The concrete that is first deposited is uncovered inside of three or four hours by jacking the form upwards. The whole form is then raised with the jacking mechanism, the men standing on top of it. The lifting movement amounts to only about ¼ in. at a time, and the form is continually filled and kept in motion. The work goes on day and night, and we build from 8 to 10 ft. in height of the wall in 24 hours, depending upon the temperature of the air. The form is about 3 ft. 6 ins. to 4 ft. high.

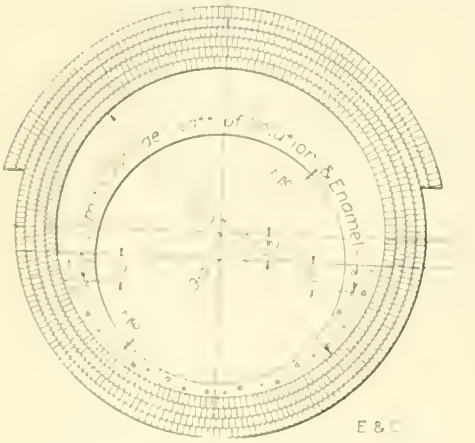
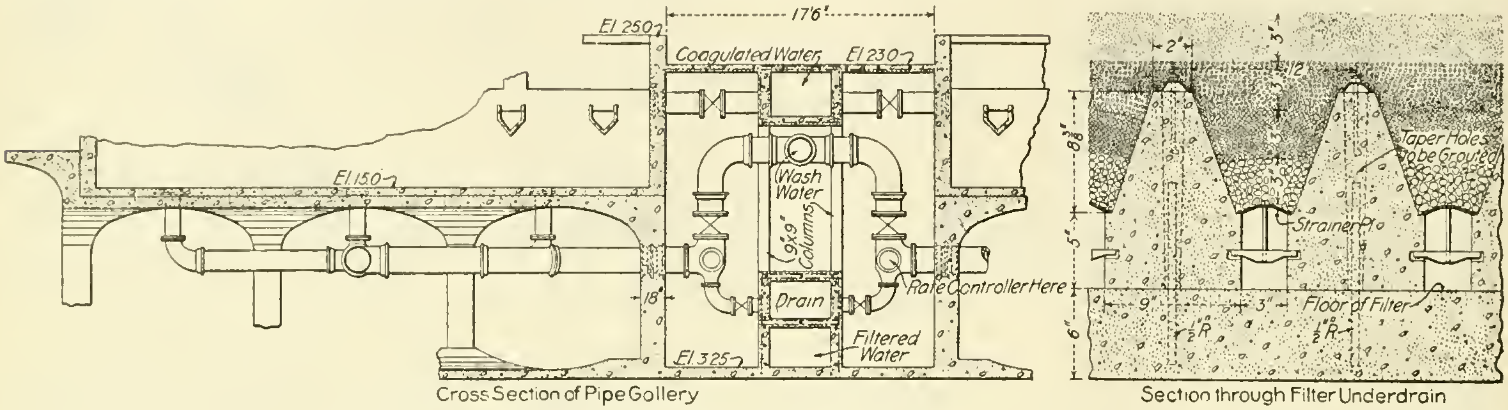


Fig. 1. Suggested Section for Laying 9-ft. Steel Pipe Within 12-ft. Brick Conduit at Baltimore, Md.

This method is employed mostly in the construction of grain bins or similar containers, and wherever we have a compartment to build which admits of using parallel vertical walls. At the present time we propose to use the method in connection with some thin walls that are to be built, instead of using plaster and expanded metal. The panel walls are to be 2 ins. thick and 12 ft. high, and it would be difficult to plaster them in the ordinary way. We propose to build the walls with moving forms, reinforce them with rods in the usual way, placing the rods in the walls as the form moves up. We shall build the chemical tanks in the same way.

The mechanism used has a pumphandle apparatus which grips the vertical rod concentrically, so that the load is carried in the center of the rod. As stated, we use 1⅛-in. rods, doweled together, and a ½ in. dowel. When the grip is passing that, it does it very easily and has no distortion, no side motion, so that the concrete holds the lower rod securely as it is setting up, and the incoming rod, of course, passing through the back, is free to come down and the mechanism clamps that way.



Cross Section of Pipe Gollery

Section through Filter Underdrain

Fig. 3. Cross Section of Reservoir, Pipe Gallery and Underdrains, Evanston Filtration Plant.

Prescribed Procedure for Laying 9-Ft. Steel or Concrete Pipe in Existing 12-Ft. Brick Water Supply Conduit at Baltimore—Old Conduit Alternately In and Out of Service.

(Staff Article.)

An interesting feature of the new water pipe line work at Baltimore, in connection with the improvement of the Gunpowder River supply, is the proposed laying of new reinforced concrete pipe lines within the existing 12 ft. brick conduit known locally as the Clifton conduit. This portion of the work consists of laying 4,125 ft. of 9-ft. and 1,240 ft. of 7-ft. reinforced concrete pipe inside of the Clifton conduit. Alternate bids also were received on 9- and 7-ft. steel pipe, but the contract was awarded to the Lock Joint Pipe Co. for the furnishing and placing of the reinforced concrete pipe. Figures 1 and 2 herewith, respectively, show the cross sections upon which bids were received for the 9-ft. steel and concrete pipe lines to be placed within the existing brick conduit.

The old brick Clifton Conduit is a gravity conduit and was not built to withstand pressure. The new conduit will be built to withstand a maximum working pressure of about 35 lbs. per square inch. It was decided to lay the reinforced concrete pipe inside the old brick conduit because it was believed to be cheaper to do this than to lay an entirely new conduit. This belief was borne out by the bids which were received. The lowest bid received for laying concrete pipe within the brick conduit, as hereinafter described, was $22.50 per foot for the 9 ft. pipe and $16.50 for the 7 ft pipe respectively. The corresponding lowest bids for steel pipe of the same sizes to be laid within the brick conduit were $34 and $30, respectively.

The Clifton conduit connects Lakes Montebello and Clifton, two storage reservoirs of the Baltimore supply system, and, under normal conditions, is in daily use, conveying the entire city consumption from the upper to the lower lake from which the city is directly supplied.

In order to maintain the water supply to the city during the installation of the inner pipe line it will be necessary to interrupt that part of the work at certain regular intervals. In general the Clifton conduit will not be out of service during a period exceeding 48 consecutive hours, but in case of emergency this period may be extended to not over 72 hours. Exact working period during which the conduit is thrown out of service to permit of work being done within it will be followed by a nonworking period equal in duration to the working period to permit of the delivery of water to the city. As the contract includes other work than that here under consideration it is expected that the contractor will arrange to divide the time of his construction forces between the work to be done within the conduit and the work outside of it, so that a minimum of lost time will result.

The draining and filling of the Clifton conduit will be done by employes of the water department. It is planned that toward the end of each working period during which work is done on the conduit, the contractor will notify the engineer at what hour it will be ready to deliver water, and the contractor will see to the removal of all tools and appliances that might be moved by the flow of water through the conduit or that offer any obstruction to the flow other than as specifically permitted. Conversely, the engineer will notify the contractor in advance at what hour the supply gates will be closed and the drain gates opened for draining the conduit.

Sanitary Regulations.—The sanitary regulations held necessary for the proper protection of the Clifton conduit are of special interest since the water supply of the city will, at intervals, traverse this conduit during the proposed construction operations here under consideration.

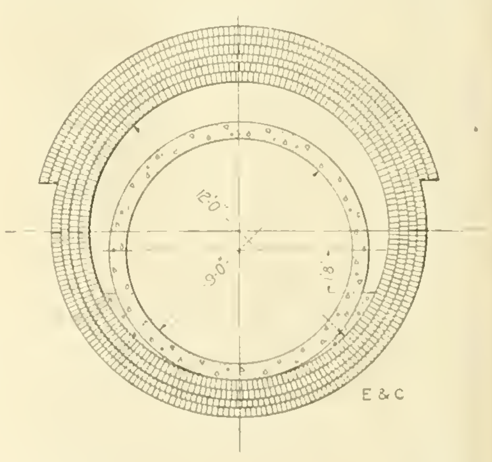


Fig. 2. Proposed Section for Laying 9-ft. Reinforced Concrete Pipe in 12-ft. Brick Conduit, Baltimore.

The contractor will provide rubber boots or shoes for all employes whose duties require them to go into the Clifton conduit. These boots or shoes will not be worn elsewhere than in the conduit and to this end the contractor will provide at all points of entrance to and exit from the conduit a suitable room or building in which the employes will be required to put on these boots immediately before entering the conduit, and to take them off on leaving, and in which the boots will be kept while not in use. Provision will be made for washing and disinfection of boots so that no contaminating substance will be carried into the conduit on them. No expectoration will be allowed nor will the committal of any act of whatever nature be tolerated that might tend to contaminate the city water supply.

STEEL PIPE WORK IN CONDUIT.

Some of the provisions of the plans and specifications for steel pipe work within the conduit are of interest despite the fact that only reinforced concrete pipe will be so employed in this instance.

The use of pipe constructed of alternate inside and outside cylindrical courses was contemplated, each course to consist of not more than two plates with a minimum width of plate of 8 ft. on straight pipe. Circumferential field joints were to have been riveted insofar as practicable, otherwise the use of boiler patch bolts would have been allowed. A minimum thickness of plate of 7/16 in. was specified. All pipe was to have been coated at the mill inside and out with one coat of Bitumastic Solution. After delivery of the pipe on the ground the interior of the pipe was to have been given one additional coat of the solution and one coat of Bitumastic Enamel. All exterior surfaces of the pipe where not encased in concrete were also to have received one additional coat of the solution and one coat of the enamel. Abrasions made in the coating during riveting operations were to have been later touched up with the enamel. A thickness of 1/32 in. was specified for the enamel.

The solution and enamel above mentioned are manufactured by the American Bitumastic Enamels Co. of New York City. These preparations are applied only by employes of the manufacturers. The quotations made for the Baltimore work may be of interest and are given herewith. The price quoted for one shop coat of the solution, complete, was ¾ ct. per square foot of surface covered, and for one field coat of the solution and one of the enamel 7½ cts. per square foot of surface covered. These prices do not include sand blasting, cleaning with gasoline, or the pickling and washing called for in the specifications, but do include a wire-brushing of the pipe prior to the application of the shop coat, whatever cleaning is necessary in the field or surfaces to receive the field coats, and the supplying of all labor, materials and tools required for the coating operations.

A 1:3:6 concrete was specified for use under the pipe within the conduit. The steel pipe would have been placed on cradles or blocks moulded in place or outside the conduit. If moulded outside the blocks would have been set in a stiff bed of 1:3 mortar, accurately to line and grade. At least one such support, properly grooved for grouting and moulded substantially to the curvature of the pipe, was specified for each ring of pipe. While placing concrete under the steel pipe in the conduit it was planned to maintain the pipe under the required water pressure or to hold it by suitably placed external bracing to the form assumed under pressure. This pressure was to have been maintained for 24 hours after the placing of the concrete.

REINFORCED CONCRETE PIPE.

As stated, reinforced concrete pipe will be placed in the conduit instead of the steel pipe above described. The thickness of concrete pipe shell will be not less than 9 and 8 ins. for the 9- and 7-ft. pipe respectively. The concrete will contain not less than 1 cu. ft. of Portland cement to each 1½ cu. ft. of sand and to each 2½ cu. ft. of stone or gravel. Maximum permissible dimension of stone is ¾ in. The reinforcing steel will not be stressed above 10,000 lbs. per square inch where pipes are under a hydrostatic pressure of 35 lbs. per square inch. Concrete will be placed under reinforced concrete pipe after the pipe has been set to line and grades on suitable wooden blocks.

The concrete pipe will be manufactured and placed by the Lock Joint Pipe Co. of New York City. Mr. Ezra B. Whitman is engineer and it is from his plans and specifications that the foregoing article has been prepared.

The Occurrence and Isolation of B. Coli in Water.

In determining the sanitary quality of a water analysts are not at present able to apply tests and measurements comparable to those used in solving engineering problems, so far as the precision and accuracy of results are concerned. In his problem the water analyst finds the factors very complex and his deductions, therefrom, resemble "a doctor's diagnosis rather than an engineering test." The methods employed at present, and their limitations, were discussed by Prof. C. M. Hilliard of Purdue University, in a paper before the recent annual meeting of the Engineering Society. The information here given is taken from that paper.

The sanitary analysis of water is chiefly concerned with the detection of waters that are dangerous to the health of the consumer; or better, in discriminating between safe and unsafe waters. The ideal bacterioscopic test would be one that would determine positively and immediately the presence or absence of disease germs. But we are very far from this ideal in the present development of the science and we have to accept more or less indirect evidence of the presence of disease producing bacteria as a basis for our verdict.

Standard procedures for the bacteriological examination of waters have been pretty generally reduced to methods of detecting specific kinds of bacteria of sewage origin; kinds associated with disease bacteria. Quantitative results frequently yield valuable information when considered together with other data but as our methods never reveal the total bacterial content but only the relative numbers that will grow to visible size in a certain time and under certain environmental conditions, and as we have been unable to approximate a numerical standard for different types of waters, and to account for fluctuations in numbers due to meterological and seasonal changes, these tests have yielded in importance to the qualitative tests. While not presuming to discard the significance of the quantitative tests with a few cursory remarks since many, especially the Germans, attach great importance to this aspect of the problem, the subject of the present article excludes the advisability of a further consideration of the question.

The remainder of the article will be confined to the question of the importance and the detection of the Bacillus coli in natural waters, without reference to the value of the finding of other associated sewage forms in this source. The problem of the significance of coli in filtered or treated water needs special consideration and is not discussed herein. The acceptance of the presence of B. coli as indicative of the pollution of water with sewage is based upon certain well established facts here briefly reviewed: In the first place, as the normal denizen of water we rarely find coli present in any considerable numbers. It is quite generally found in nature, in dust, soil and upon cereals. It is present in normal, unpolluted waters only in very small numbers. In ground waters it may occasionally be found in 100-200 cc. samples, rarely in 10 c. c. In surface waters it is frequently isolated from 10 c. c. but is rarely found in smaller samples of unpolluted waters of any kind.

In the second place, we know beyond a doubt that the colon bacillus is the normal and constant inhabitant of the large intestine of man and warm blooded animals. We unfortunately have no way of distinguishing the particular source from which an isolated bacillus may come.

Finally, we have definite and delicate methods of demonstrating the presence or absence of B. coli in any given source and experimentally and practically we have shown that this organism may be constantly isolated from water which has received sewage even though in enormous dilution (1:1,000,000 to 1:10,000,000).

If the first two propositions are true, concerning the distribution of coli in nature, then the value of our tests depends fundamentally upon the third proposition, or the accuracy of our methods of detecting this organism in water, and upon the establishment of a quantitative limit, above which or below which a water may be condemned or approved. With regard to this latter point, we find water analysts very cautious and conservative in their opinion as to where the dividing line should come and we may never hope for an empirical and rigid rule as a standard because of the very nature of the problem as here outlined. The coli content of a water is, we believe, the most valuable single fact that we can obtain and together with the other evidence; chemical tests, knowledge of the origin of the water, its sanitary surroundings, etc., it forms the nucleus about which we mould our conclusion. Dr. Houston, perhaps the greatest English authority on the subject, has reiterated his opinion of several years ago. He believes that a water showing no evidence of coli in 100 c. c. shows no evidence of objectionable contamination; present in 100 c. c. but not found in 10 c. c. "not absolutely free from the suggestion of being potentially if to an almost negligible extent, dangerous to health"; present in 10 c. c. but not in 1 c. c. "contamination presumably objectionable in character, and therefore to be regarded with some suspicion"; present in 1 c. c. but absent in 0.1 c. c. shows "obvious indications of pollution" and is not safe "in the absence of contra-indications of a convincing kind." The French bacteriologists attach the same significance to B. coli as an indicator of pollution and generally fix the border line of doubt between the limits of coli in from 1 c. c. to 10 c. c. while its presence in 1 c. c. and 0.1 c. c. is recognized as dangerous. Prescott and Winslow, after a careful review of all the work and upon the basis of a broad experience, make the following statements in regard to the significance of the quantitative aspect of coli in water:

> Altogether the evidence is quite conclusive that the absence of B. coli demonstrates the harmlessness of a water as far as bacteriology can prove it. That when present, its numbers form a reasonably close index of the amount of pollution . . . It may safely be said that when the colon bacillus is found in such abundance as to be isolated in a large proportion of cases from 1 c.c. of water, it is reasonable proof of the presence of serious pollution.

Before considering our methods, and the reliability of our methods of B. coli isolation, it may be well to indicate briefly what the characters and properties of the organism under discussion are. B. coli refers to a group, rather than to a single definite organism. In order to be admitted to the typical group, an organism must be a short, non-spore-forming, Gram negative, bacillus, usually motile and should exhibit the following characteristic biochemical characters when grown upon various artificial culture media: They uniformly ferment dextrose and lactose media with acid and gas production; coagulate litmus milk and reduce the litmus; do not liquify gelatin; give a positive reaction in esculin media and generally produce indol in peptone solution. Of all the tests, none is more specific than the capacity to ferment sugars with gas, particularly the lactose. This is, par excellence, a test for bacteria of intestinal origin and we find that most of our special "presumptive tests" hinge on this reaction.

Isolation, or the obtaining of the desired organism in pure culture may be done directly from the original specimen, or indirectly by first inoculating suitable and selective liquid media with various dilutions of the water. The direct method is the quickest but is generally applicable in the examination of highly polluted waters because of the limit to the amount of water that can be plated and more serious, because of the danger of overgrowth of the sewage forms by other bacteria. The principles involved in either the direct or indirect method are fundamentally the same: the providing of a suitable and selective pabulum for the growth of the desired organisms and the presence of substances or devices that will give reasonable conviction of the presence of sewage bacteria.

The plate method of isolation, as stated, finds its chief drawback to be that of overcrowding. The litmus lactose agar media is the oldest and still the most generally used of all the solid media as a presumptive coli test. Coli on such plates form definitely shaped colonies fermenting the sugar and turning the indicator pink. The Endo medium (fuchsin-lactose-agar, the fuchsin reduced by sodium sulphite) and the Drigalski and Conradi medium (a specially prepared lactose agar with crystal violet added) inhibit the growth of many saprophytic bacteria and produce characteristic colonies from the growth of coli. I have been working upon a new medium involving the same selective principle. Churchman has shown that the Gram positive organisms refuse to grow in the presence of the gentian violet dye and as coli and its allies in sewage are chiefly Gram negative its use in media for water analysis is apparent. It has been found that in litmus-lactose-violet-agar we may cut the total counts down from 50-80% and still obtain growth of all the coli present. So far as our work has gone the presence of the violet stain seems to have no inhibitive action upon either coli or typhoid so that it would make possible the plating of from one to one and a half times as much water as on the ordinary lactose plate without jeopardizing the test for the specific sewage bacteria.

The liquid media have the advantage of having no quantitative limit as to the amount of water that may be examined. Preliminary enrichment or growth in a highly favorable medium is sometimes resorted to to encourage the revitalization of certain sickly or attenuated coli, but for practical purposes this procedure is usually omitted.

From all the liquid media that have been proposed and used we find a remarkable uniformity of opinion in favor of ox bile or bile salt media with the addition of sugars: dextrose or lactose. Bile salts act as highly selective inhibiting agents to the vast majority of saprophytic bacteria while coli and many other sewage bacteria grow readily in bile and the coli ferment the sugar present with gas production in 48 hours. This test is more specific and more reliable than any other known test. It has been shown by Jordon "that B. coli itself is inhibited in a marked degree in media containing bile." Attenuated coli is severely restricted in bile but the Standard Methods Committee of the American Public Health Association advises that from a practical standpoint "it is best to discount the presence of attenuated B. coli."

Whether we should depend upon the fermentation of dextrose or lactose in the presence of bile salts seems to me to be still a mooted question. Many strains will ferment the nonsaccharide that fail to ferment the disaccharide in bile and higher results would be expected with the dextrose. However, some forms of bacteria other than coli ferment dextrose so that the use of the less complex substance is not quite so specific. English opinion favors the use of dextrose while Americans for the most part seem to recommend the use of lactose bile. We are, it seems to me, requiring a good deal of the bacteria that are weakened by long sojourn in water, to expect them to grow in the presence of a substance that is an active inhibiting agent even to many vigorous coli and also expect them to split the more complex sugar. We believe that re-inoculations into lactos bile should always be made and in this case the original use of dextrose bile should give us somewhat more delicate results.

To recapitulate: We have gradually come to the opinion that the presence of B. coli in water in certain numbers is strong presumptive evidence of pollution of sewagic origin to be used in connection with other data in forming our verdict as to the safety of a water. The isolation of coli from water is best made by direct inoculation of various dilutions of the original specimen into the highly selective bile—dextrose or lactose—and a subsequent transfer into lactose bile and preferably, into other confirmatory media.

The Electrolytic Corrosion of Iron in Soils.

The term "electrolytic corrosion" is most frequently used to indicate corrosion caused by the discharge of an electric current which enters the metal from external sources. During recent years, however, the theory has been widely accepted that all corrosion in water solutions is essentially electrolytic in its nature, and in consequence there have come into use a variety of terms, such as "galvanic action," "stray current electrolysis," "self-corrosion," etc., to distinguish between cases of corrosion originating from different causes. Thus, corrosion of buried iron may be due to galvanic action caused by physical or chemical differences between adjacent points on the surface of the metal, to the presence of foreign substances in the soil, such as coke cinders, iron oxides, etc., which set up local galvanic action, or it may be due to the discharge of electric currents that have entered the structure at some remote point. In the present discussion the terms "electrolysis" and "electrolytic corrosion" are used to designate corrosion caused by the discharge of electric current which has entered the metal from some outside source, while all other forms of corrosion in which the electric currents originate within the corroding system itself from whatever cause are referred to as "self-corrosion." It should be pointed out at the outset, however, that these two general classes of corrosion are by no means independent of each other, since the presence of either kind of corrosion generally affects in marked degree the nature and extent of the other under a given set of conditions. This mutual influence is such as to be of considerable practical importance, and in particular it often greatly increases the difficulty of obtaining trustworthy experimental data in regard to electrolytic corrosion proper.

The U. S. Bureau of Standards has been for some time conducting a general investigation of the subject of electrolysis and electrolysis mitigation, and a recent paper has been published dealing with the fundamental laws governing electrolytic corrosion under practical conditions, and relating to self-corrosion only insofar as it is necessary to distinguish between the two classes. The paper was prepared by Burton McCullom and K. H. Logan, Physicists of the Bureau of Standards, and is called Technologic Paper No. 25. The subject of the prevention of electrolytic damage is referred to only incidentally, when occasion requires, in order to interpret the significance of the results obtained. The general conclusions drawn in the paper mentioned are here given, with only sufficient introductory matter to make clear the nature of the problems studied. The full paper contains numerous tables and diagrams to which we cannot give space at his time.

In studying the phenomena of electrolytic corrosion in soils under practical conditions many variables are encountered which tend in greater or less degree to affect the results. Among these may be mentioned the current density at the surface of the metal, the moisture content of the soil, the presence of oxygen either in the gaseous state or dissolved in soil waters. The latter not only affects the rate of corrosion but also affects the character of the end products of the reactions, and thus to some extent has a bearing on the question of diagnosing the cause of particular cases of corrosion. The temperature of the soil is also important, particularly because of its effect on the current flow. In the case of iron, the formation of oxids as a result of the initial corrosion may complicate matters because of their possible action in stimulating galvanic action. Other factors, such as the mechanical and chemical properties of the soil, the depth of burial of the metal, the limitation of current flow due to polarization, the formation of high resistance films on the surface of the metal, and the pitting of the surface, due to a variety of causes, may likewise act to increase or decrease the rate at which damage may progress, and therefore require special investigation. It is these factors that are dealt with in the report, and while the investigations have in most cases not yet been completed, we believe that the tentative conclusions of the investigators, relative to the effect of the several variables above enumerated, will be of interest.

GENERAL CONCLUSIONS BASED ON EXPERIMENTAL DATA.

The following are the more general conclusions that have been drawn from the experimental data published in the full paper:

1. The current density has a marked effect on efficiency of corrosion of iron in soils, the efficiency of corrosion being, in general, greater the lower the current density. In saturated soils the corrosion may vary between 20 per cent and about 140 per cent for the range of current densities between 5 milliamperes and 0.05 milliampere per square centimeter.

2. Moisture content in the soil also has a marked effect on efficiency of corrosion, the corrosion efficiency being in general greater with increasing moisture content up to saturation of the soil. Beyond this point increased moisture content has comparatively little effect.

3. Temperature changes within the limits commonly met with in practice have no important effect on corrosion efficiency.

4. The depth of burial of pipes has no direct effect on corrosion efficiency provided other conditions remain constant. In practice, however, the moisture content, current carried by the pipes, and various other factors which affect corrosion efficiency will vary with depth, so that indirectly differences due to depth may be noted.

5. The amount of oxygen present has no appreciable effect on the efficiency of corrosion in the case of iron immersed in liquid electrolyte.

6. Corrosion efficiency of iron embedded in earth is always greater in open vessels than in sealed vessels.

7. The amount of oxygen present has a marked effect on the final products of corrosion. If the corrosion is rapid and the supply of oxygen small, there will be a preponderance of magnetic oxid, while if the rate of corrosion is low and the supply of oxygen abundant the ferric oxid will predominate. Owing to the fact that the supply of oxygen around pipes buried in earth is always more or less limited, the character of the oxids formed gives some indication as to the rate of corrosion, and thus indirectly the cause of the corrosion, if local conditions are properly considered.

8. There is no material difference in the efficiency of corrosion shown by the various kinds of iron commonly used in the manufacture of underground pipes.

9. The fact that a given chemical tends strongly to inhibit either self-corrosion or electrolytic corrosion in liquids is no indication that it will materially retard electrolysis of iron embedded in soils.

10. Pitting of iron embedded in soils is affected not only by a nonhomogeneous condition of the iron or soil, but also by the chemicals contained in the soil.

11. The efficiency of corrosion was found not to be a function of the voltage except in so far as the current density may be affected. Voltages as low as 0.1 and 0.6 volts showed practically the same efficiency of corrosion as 5 to 10 volts or higher.

12. Corrosion tests on a large number of different kinds of soil from widely different sources with average moisture content and moderate current density indicate that corrosion efficiencies between 50 and 110 per

cent may usually be expected under most practical conditions.

13. The resistance of soils varies throughout a very wide range with variations in moisture content, the resistance of the comparatively dry soil being of the order of several hundred times the resistance of the same soil at about saturation. Above saturation, increase in moisture content has but little effect on the resistance of the soil.

14. Because of the great variations in resistance of earth with moisture content, voltage surveys should not be made at times when the earth is extremely dry.

15. The resistance of the soil varies greatly with temperature within the ordinary range encountered in practice. In the case of the soil tested the resistance at 18° below zero centigrade was over 200 times as great as at 18° above zero centigrade. Even at about freezing temperature the resistance will be several times that at summer temperatures. This not only has an important bearing on the magnitude of the electrolysis trouble that may occur at different seasons, but it also indicates that where practicable voltage surveys should not be made when extremely low temperatures prevail.

16. The experimental results given in the paper have an important bearing on the subject of electrolysis mitigation through the limitation of voltage drop in the negative return. For some years the chief means of preventing trouble from electrolysis in certain foreign countries has been the limitation of the permissible voltage drop between any two points on the return circuit. In some places the limit has been placed on the maximum voltage during peak load, whereas in other cases the average voltage for 24 hours has been the determining factor. It will be evident that if the total amount of damage which results is proportional to the average current, then the limitation of the average voltage would be more logical than the limitation of the peak-load voltage, since in the former case the cost of meeting the voltage limitation in any given case would be proportionate to the danger involved irrespective of the station load factor; whereas if the voltage at peak load is the determining factor the cost of complying with the requirements depend not only on the danger involved, but on the load factor of the system, and the poorer the load factor the greater its cost will be. It appears from the data presented in the paper that the rate of damage does not increase as fast as the voltage increases, because of the tendency toward lower corrosion efficiencies at higher current densities. This indicates that, with a given average all-day current the actual amount of electrolysis that would occur would be less with a bad load factor than with a good load factor, and hence points to the undesirability of penalizing a high peak of short duration. It would appear more logical, therefore, in so far as the damage itself is concerned, to make the average all-day voltage the basis of the limitation rather than the voltage at time of peak load.

Notes on the Proper Use of Chlorinated Lime for the Disinfection of Drinking Water.

The use of chlorinated lime for the disinfection of public water supplies has been discussed from many points of view in former issues of this journal. The advantages of this method of disinfection have, perhaps, been sufficiently emphasized. The method has now been in use long enough so that some of the more common errors in its application may at this time be discussed to advantage. There is now a better understanding, moreover, of the types of water which are best suited to chlorination, and overdosing with chlorine has received considerable attention. These latter considerations are discussed in the present article, the matter given being taken from an article by M. L. Holm and F. R. Chambers, of the Michigan State Bacteriological Department, published in the Michigan Public Health Journal.

METHODS OF APPLYING.

In the practical application of hypochlorite, the common practice is to prepare a solution of about ½ per cent strength, that is, about 5 lbs. of hypochlorite to 100 gals. of water and feeding this solution in proper proportion into the public supply. With an average product a dilution of a ½ per cent solution at the rate of 1-6,600 will give about 0.3 parts available chlorine per 1,000,000 parts of water or approximately the amount required for slightly contaminated waters from the Great Lakes system. In practical work, however, the theoretical results are often not obtained. Different lots of hypochlorite vary in composition and the last portions from a container are generally weaker than that from a freshly opened container. Several plants in Michigan have from time to time submitted samples of powder for examination showing such reduction. This, however, does not appear to be the most important source of error. Imperfect solutions are often obtained and they in turn deteriorate in the storage tanks. We recently had occasion to examine a hypo solution from one of our Michigan city plants and found that the fresh stock solution was about 20 per cent lower in available chlorine than it should theoretically have been from the amount of hypo added. Another 10 per cent deterioration was found to have occurred after two weeks' storage in the solution tank. For accurate and reliable work, such unknown factors must necessarily be eliminated as much as possible. It would appear that it is necessary to make frequent titrations of the solutions in the solution tank in order to have any very accurate idea of just how much available chlorine is being used. This is a rather simple procedure and with a little practice could be done and should be done by some one at the plant. In addition to this, bacteriological examinations should be made regularly to check the results.

TYPES OF WATER SUITED FOR CHLORINATION.

Successful results with chlorine treatment may be obtained only when the water is not too highly contaminated, clear and of uniform organic composition. Turbid waters must be filtered in order to secure successful results. Where water supplies are subject to intermittent increase in the amount of contamination, chlorination is generally a failure, because the amount of chlorine necessary for efficient disinfection during high contamination will be objectionable to the consumers during periods when contamination is low or absent, and the greatest permissible amount used when contamination is low will be too quickly neutralized and ineffective when contamination is materially increased.

We are often asked concerning chlorine treatment of well waters. Contaminated well waters respond to the action of chlorine as do surface waters and if the water is clear and the contamination uniform the treatment may be successful. However, the amount of contamination present in unsafe wells is usually subject to extreme variations and not at all suited for successful chlorine disinfection.

OVERDOSING WITH CHLORINE.

In some instances it has been found practically impossible to obtain sufficient bacterial destruction without adding so large a quantity of chlorine that the water becomes unfit for drinking purposes. Where such quantities are required it is possible to introduce further treatment that will remove the excess of chlorine and eliminate the objectionable odor and taste.

The methods that have been advocated for the removal of excesses of chlorine are storage, aeration and the addition of non-toxic chemical reducing agents.

It has been pointed out that the amount of chlorine required to disinfect a given water represents only a small fraction of the actual quantity of reducing substances in that water and it may be theoretically assumed that if sufficient time be allowed chlorine will continue to be absorbed up to the saturation point of the reducing substances present. Many water works engineers have at times noticed a marked taste of chlorine at the pumping station when no such taste was perceptible at various points in the city supplied.

Aeration materially hastens the disappearance of excesses of chlorine probably by direct evaporation of the uncombined gas. The chemicals available for eliminating the excess of chlorine are chiefly sodium sulphite and sodium thiosulphate. Of these the latter has the advantage of being cheaper and more stable. The actual increase in cost of chemicals where sodium thiosulphate is used will vary with the different waters, but the cost of neutralizing will not exceed one-half of the cost of the chlorinated lime employed.

Under ordinary conditions the action of chlorine upon bacteria takes place very rapidly and the maximum bacterial destruction will generally be reached within five minutes. Where neutralizing chemicals are employed, however, it is advisable to allow longer exposure in order to be assured that the chlorine has been thoroughly distributed and that the action is complete. It has therefore been recommended that a period of at least 10 or 15 minutes should interevene between the chlorination and the subsequent chemical neutralizing. This double chemical treatment may in some instances involve a somewhat difficult engineering problem in the mechanical construction, but when installed will possess the advantage that chlorination to the point of satisfactory disinfection may always be carried out. The quantities of sodium thiosulphate required to neutralize even a considerable excess of chlorine will leave no objectionable by-products and moderate excesses of the chemical itself will be tasteless and unobjectionable.

In conclusion, there can be no doubt but that we have in chlorine a cheap and efficient agent for the practical disinfection of drinking water when properly applied.

The germicidal effect of chlorine, whether used as anhydrous chlorine or hypochlorite, is essentially the same. Chlorine acts rapidly upon most vegetative forms of bacteria, but is selective in action and does not destroy bacteria of the tubercle group or spore forms.

The use of chlorine cannot be considered a substitute for filtration. Where a water is low in organic matter, clear and only slightly polluted, the use of the chlorine or hypochlorite treatment alone may be sufficient. In combination with filtration, however, chlorine should give excellent results. Filtration alone gives at best only a fractional purification and unlike the chlorine treatment the bacterial efficiency is in no way selective. By combining the use of chlorine and filtration there may be substantial economy in the first cost of the filtration plant because higher rates of filtration may be used and the required filter area proportionately reduced.

Some Examples of Poor Design and Improper Operation of Small Water Purification Plants.

The fact that small water purification plants are frequently abused in their operation is, as we have before stated, well known to engineers. It is with a view, therefore, of supplying engineers with additional data on this point, for use as illustrative examples in their efforts to impress upon city officials the importance of competent operation of such plants, that we are here publishing an abstract of a paper recently presented by H. P. Letton, Sanitary Engineer of the United States Public Health Service, before the New England Water Works Association.

It very frequently happens that when a small water purification plant is installed in connection with a water works system which has been in operation for some time, perhaps for several years, that the man who has had charge of the pumping station takes on the operation of the purification plant as an additional duty. It is very uncommon, indeed, for men of this type to understand the process of water purification, even in its fundamentals. In connection with the discharge of his duties Mr. Letton has had an unusually good opportunity to observe the more common faults in the design and operation of the type of plants here under consideration and he mentions the following examples.

EXAMPLES OF POOR DESIGN.

The worst example cited related to a plant of 1,000,000 gals. daily capacity. The water supply, obtained from an artificial lake, was highly colored, due to passing through cedar swamps. The water flowed from the lake to the coagulation basins by gravity. In this basin, which was merely a wooden tank, a period of less than 20 minutes ordinarily was employed for coagulation and sedimentation purposes. This period was still shorter in times of high water consumption. Moreover, the coagulation basin was set at such a high level that in times of low water in the lake it was necessary to by-pass raw water around the basin in order to maintain the necessary supply. The filter plant consisted of four rapid sand, circular, wooden tank filters, operated by gravity and fed by a gravity flow from the coagulation basin. The filters were not equipped with either loss of head gages or rate controllers. The beds were not agitated during washing because the rakes provided for that purpose were fitted up for water motor drive and the necessary water pressure for actuating this motor was lacking. After leaving the filter the water passed to a suction well giving less than 30 minutes' storage. A small solution pump, for pumping a solution of sulphate of alumina into the raw water just above the coagulation basin, was supposed to be driven from the main shaft which drove the pumps of the distribution systems. It was learned, however, that this solution pump was normally out of service and that for the greater part of the time no coagulating chemical was injected into the raw water. Upon making an analysis of the raw water it was discovered that it was slightly acid, probably due to humic acid taken up by the water in passing through the cedar swamp previously mentioned. Under these circumstances the addition of the sulphate of alumina would have been useless. Analyses were made of both the raw and filtered water and the improvement shown by the filtered over the raw water was but slight. The color of the filtered water was at times at 200 p.p.m.

Mr. Letton investigated two other plants in which acid waters were being treated with sulphate of alumina only. Among the small plants inspected rate controllers and loss of head gages were seldom found, and the methods employed in controlling the chemical feed were in most cases very crude. Only a few calibrated orifice boxes were found in service. Two gravity rapid sand filters, which heated waters of high organic content and frequently turbid, were not operated in conjunction with coagulation basins. This led to too frequent washing of the filter sand and the bacterial efficiency of the filters was correspondingly lessened, and operation costs were increased.

One large pressure filter was found treating a turbid water without sufficient time for coagulation. The effluent from this filter was frequently turbid and sometimes contained aluminum hydrate. Another plan, a slow sand plant, had less than 1 ft. of filtering sand and the capacity of the accompanying clear water well was about a half hour's supply. The rate of filtration consequently fluctuated exactly as the demand and almost no advantage resulted from filtration.

At one rapid plant raw water was used for filter washing because the clear water basin was so small. Still another plant was not designed so that it could filter to waste. Accordingly when it was necessary to gain access to the strainer system the dirty water in the bed was discharged into the clear well.

EXAMPLES OF FAULTY OPERATION.

Although good results in rapid sand plants are so largely dependent upon the proper use of the coagulant, Mr. Letton observed great ignorance on this point on the part of operators of small plants. At several plants the operator measured alum by the pailful per day, without knowing the weight used or the proportion to the water treated. A few operators stated that increased the amount of alum somewhat when the raw water was turbid, but the increase was governed only by the operator's imperfect sense of proportion and not by precise measurement of any kind. In only a few plants inspected was it found the practice to regulate the dosing by the results of making alkalinity and turbidity tests. Only a few of the operators could accurately make the alkalinity tests. Only two plants of less than 20,000,000 gals. daily capacity maintained laboratories, and even in these the chemical and bacteriological tests were made under the direction of non-resident specialists, the operators who made the tests being unable to interpret them or to use them as a guide in operating the plant.

At one plant the operator was too lazy to wash the filters sufficiently often and they clogged badly. This decrease in filtered output was made up by by-passing raw water around the plant. At another plant the beds were washed too often and the bacterial efficiency was low and the cost of operation was high. At this plant over 10 per cent of the water filtered was being used for wash water.

The International Engineering Congress of 1915.—Under the auspices of the American Society of Civil Engineers, the American Institute of Mining Engineers, the American Society of Mechanical Engineers, the American Institute of Electrical Engineers and the Society of Naval Architects and Marine Engineers, there will be convened at the Panama-Pacific International Exposition, in 1915, an International Engineering Congress.

Engineers in all parts of the world have signified their intention of attending and the delegation from Great Britain promises to be large in point of numbers and great in distinction.

The president and secretaries of the five organizations above named together with eighteen other members constitute the official body which will assemble and conduct the congress. Colonel George W. Goethals, chief engineer of the Isthmian Canal Committee, has consented to act as honorary president of the congress.

Rivers and Harbors

A Study of Reservoir Control as a Means of Flood Prevention at Columbus, Ohio.

One of the possible means of preventing flood danger at Columbus, Ohio, was considered by the engineers, John W. Alvord and Charles B. Burdick of Chicago to be dam construction on the Scioto and Olentangy rivers above the city. In their final report these engineer's submit their consideration of reservoir flood prevention in a manner which is instructive of the value and limitations of this method. For this reason and also because it furnishes an example of procedure in making flood prevention reservoir studies we submit somewhat condensed, this portion of the report.

TYPE OF RESERVOIR FOR FLOOD CONTROL.

Reservoirs for flood protection may be divided into two classes. There are controlled reservoirs, so constructed that they may be filled or emptied by means of gates, operated by attendants and dependent for their usefulness upon the skill and discretion with which they are operated. Such reservoirs are largely used for the purpose of improving the low water flow of the rivers immediately below them, either for water power or irrigation purposes, or, in some cases, for flood control and for domestic water supply. Of the latter type is the reservoir constructed for the Columbus water supply, known locally as the Storage Dam.

A second type of reservoir used especially for flood control is so arranged that in the base of the dam there is an opening of sufficient size so that all the ordinary flow of the river passes freely through without detention and where occasion requires it, small floods up to a predetermined amount can likewise be passed. Such a detaining reservoir, therefore, brings its storage into play whenever the flood flow of the river exceeds this predetermined amount, and the excess of such flood flow will be impounded in the reservoir until it is completely full, at which time the surplus, if any, can pass over the spillway at the top while the opening in the bottom, being all the time in active use is emptying the predetermined amount of flow into the ordinary channel below. A reservoir of such character obviously should be empty except in time of great flood. If sufficient opportunity offers it sometimes can be so arranged that it may not be filled except in time of the greatest flood known to occur in its river valley. Under such circumstances, for many years at a time the reservoir will be empty, and the land, if cultivated at all, can be cultivated whenever its fertility is sufficient to make this desirable. A reservoir of sufficient capacity can be made to stop only that portion of the flood which would create the maximum flow immediately below its site, thus reducing the gage heights at the critical period.

Inasmuch as large floods coming out from different tributaries have different time combinations in reaching maximum flow it is not always possible closely to predetermine to just what extent the maximum will be reduced in every flood, but with surplus storage capacity, it is safe to rely upon effective crest reduction by a design based upon careful study of the particular watershed and its past flood history.

POSSIBILITIES OF RESERVOIR CONTROL AT COLUMBUS.

It very early developed in the study of the flood situation at Columbus that there existed an opportunity to create storage reservoirs at a reasonable cost. Both the Scioto and the Olentangy rivers for some 20 to 25 miles above Columbus, flow through rock gorges with steep sides ranging all the way from 50 to 100 ft. in depth, and from 500 to 4000 ft. in bottom width. Land in the bottom of these gorges is not particularly fertile, and subject as it is to overflow, does not have a large value. Few owners have had the temerity to establish themselves in these bottoms, and such few houses and barns as exist there are not of great value.

In the rock gorge section of these rivers there are few tributaries so that a maximum amount of storage can be obtained with a minimum of land to be acquired. Opportunity exists at favorable points to construct dams on solid rock foundations and also extending on both sides into the rock, without having to be built of excessive lengths, or in any especially costly manner, so that the maximum of strength and durability can be obtained.

The location of some of these reservoirs is such that they will be effective in reducing flood heights not only at Columbus, but also at Delaware, and to some extent as far south as Chillicothe. The greatest efficiency, however, of such reservoirs in reducing flood heights is in the reach of the river immediately below them. On both rivers above these rock gorges the aspect of the country changes, it becomes more level and rolling.

and the rivers both resume the character of an ordinary alluvial stream in a fairly level country where no such opportunities exist for a large amount of reservoir storage without excessive expenditure.

An opportunity such as this could not be properly neglected when considering the flood protection of Columbus.

Flood Rate Reduced to (Sec. ft)	Percent Reduction Flood Rate	Acre Feet of Storage Required	Flow at which Reservoir should begin to fill (Sec. ft)	Length of time filling (Days)	Length of time emptying (Days)	Length of time filling and emptying (Days)
60,000	25%	20,000	60,000	$\frac{16}{24}$	$\frac{23}{24}$	$1\frac{15}{24}$
50,000	37%	38,000	50,000	$1\frac{2}{24}$	$1\frac{22}{24}$	3
40,000	50%	74,000	40,000	$2\frac{4}{24}$	$2\frac{13}{24}$	$4\frac{17}{24}$
30,000	62%	128,000	30,000	$3\frac{4}{24}$	$4\frac{1}{24}$	$7\frac{5}{24}$

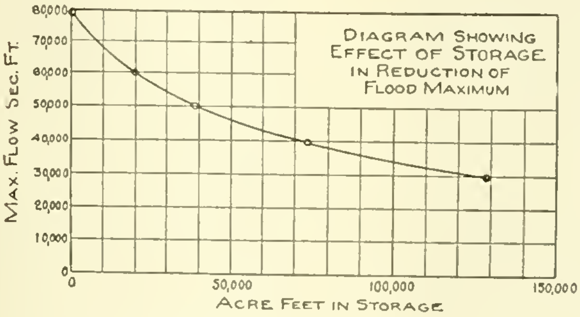


Fig. 1. Reservoir Capacities Required to Reduce Maximum Flood Rate of 1913 to Various Other Flood Rates.

Maps of the United States Geological Survey afforded opportunity for a rapid and fairly complete investigation, and detailed surveys made at a few of the best sites gave the data required for the approximate cost of erecting stable masonary dams of the detention type, which would act only in case of great floods. Four sites were thought especially worthy of investigation. Upon the Scioto, immediately above Dublin, there is a location where a dam could be built storing nearly 80,000 acre feet of water, and requiring the acquisition of about 5,440 acres of land. The crest of this dam would be about 100 ft. above the low water of the river, and the total length of the masonry about 2400 ft. along the top. An earth embankment on either end of the dam will confine the water which flows over the crest of the spillway. Should this dam ever be completely filled in its function as a flood regulator, the impounded water would reach back nearly to Bell Point.

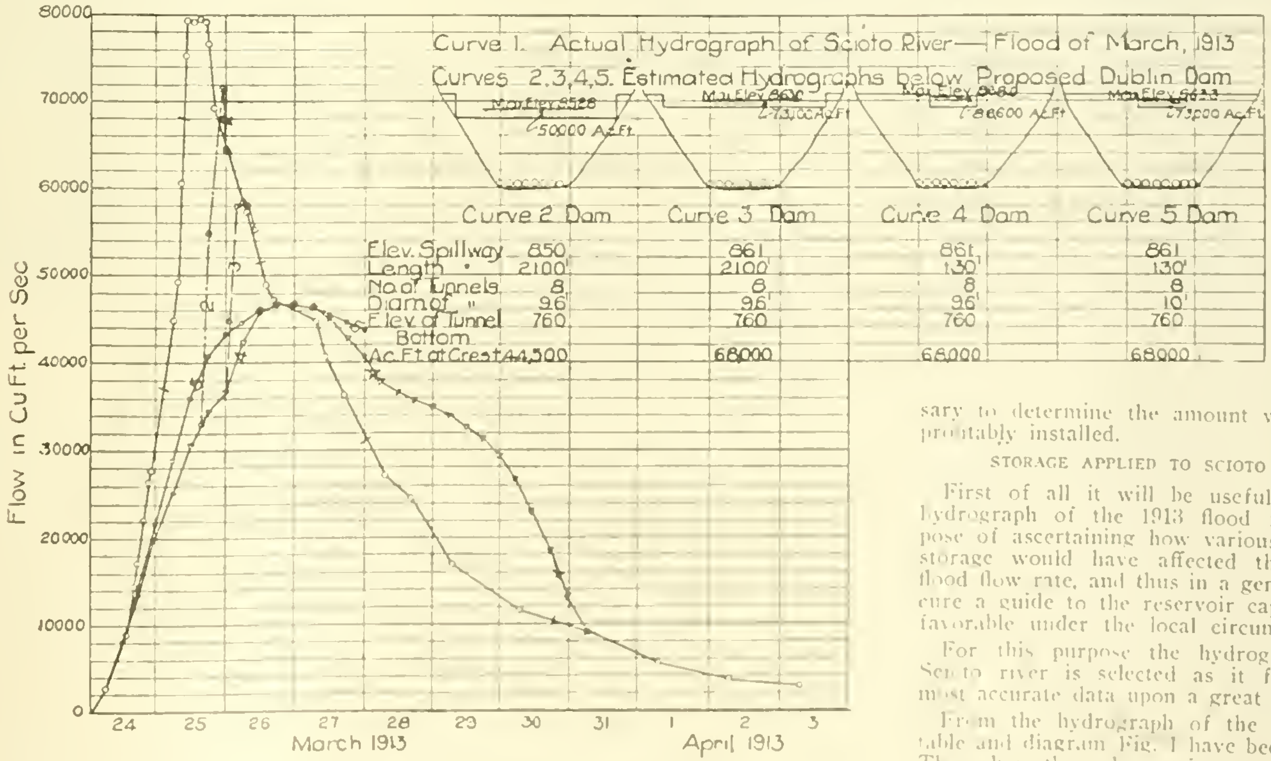


Fig. 2. Hydrographs of Sicoto River Below Dublin Dam, Showing Effect of Various Dams and Spillways in Reducing Flood of 1913.

Large openings in the bottom of this dam would pass the smaller floods and the reservoir would only come into use once in a long series of years when there would come a great flood whose maximum flow it would be desirable to control.

The total cost of this dam including land, is estimated at $2,431,200, or $30.79 per acre foot of storage secured.

A second site for a reservoir on the Scioto is found a little south of the crossing of the C. C. C. & St. L. Railroad near White Sulphur Station. Here a masonry dam could be built in the rock gorge, having a height of 50 ft. above low water, and a spillway length of 1000 ft. This dam would store 44,000 acre feet of water. It would be necessary to purchase about 11,840 acres of land of a much better class than that described for the Dublin site. This reservoir would be more expensive (costing $56.69 per acre foot stored) than the Dublin dam, its total cost being $2,494,500.

Upon the Olentangy a suitable site has been located at a point in the rock gorge west of the village of Flint, where a dam 62 ft. in height could be built, having a spillway crest 1,350 ft. in length, and storing 39,000 acre feet of water, at a cost of $1,431,700, or $36.69 per acre foot stored. This reservoir, if ever filled, would reach back to a point a little below the power house of the Columbus, Delaware & Marion Interurban line, about three miles below Delaware.

Above Delaware another site for a dam was located about a mile north of the Delaware water works. At this point the river runs in a deep narrow valley. A dam 57 ft. in height above low water, having a spillway of 130 ft. in length, of concrete masonry, would store about 50,650 acre feet of water when full, the cost of the dam being $1,146,200, or $22.63 per acre foot of storage. About 6,900 acres of land would be required for this latter reservoir.

The total amount of storage, therefore, found to be available in these practicable sites is 212,665 acre-feet, and the total cost $7,503,700, or an average per acre-foot stored of $35.30.

In studying the effect of such storage as this upon the flood heights of Columbus, it has been found that there is more storage than can be economically useful in controlling flood heights. Some analysis, therefore, is necessary to determine the amount which can be profitably installed.

STORAGE APPLIED TO SCIOTO FLOOD.

First of all it will be useful to test the hydrograph of the 1913 flood for the purpose of ascertaining how various volumes in storage would have affected the maximum flood flow rate, and thus in a general way secure a guide to the reservoir capacities most favorable under the local circumstances.

For this purpose the hydrograph of the Scioto river is selected as it furnishes the most accurate data upon a great flood.

From the hydrograph of the Scioto river, table and diagram Fig. 1 have been computed. They show the volume of storage that would have been necessary to reduce the maximum flood rate of 80,000 sec.-ft. to the smaller rates of 60,000, 50,000, 40,000 and 30,000 cu. ft. per second, respectively. The computation is made upon the theory that the excess flood water would be diverted to the storage reservoir at such time and in such amount as would most favorably affect the flood rate; that is, in considering a reservoir to reduce a flood to 60,000 cu. ft., the reservoir would begin to fill when the river reached a flow of 60,000 ft., and would continue until the river had again fallen to 60,000 ft., after which the accumulated storage would be discharged in such amount as would not increase the flood rate above the maximum fixed. Obviously, such manipulation is not a practical matter, but the computation serves to indicate the relative usefulness of various storage volumes, and it indicates that about 20,000 acre-feet in storage would have been effective in reducing the maximum rate in 1913 on the upper Scioto about one-fourth. 74,000 acre-feet would have reduced the maximum flood flow about one-half, and 130,000 acre-feet five-eighths. The computation further serves to show that the greatest percentage of effect in reduction of flood rate is secured by a comparatively small reservoir. The table referred to also shows the length of time theoretically required in filling and emptying.

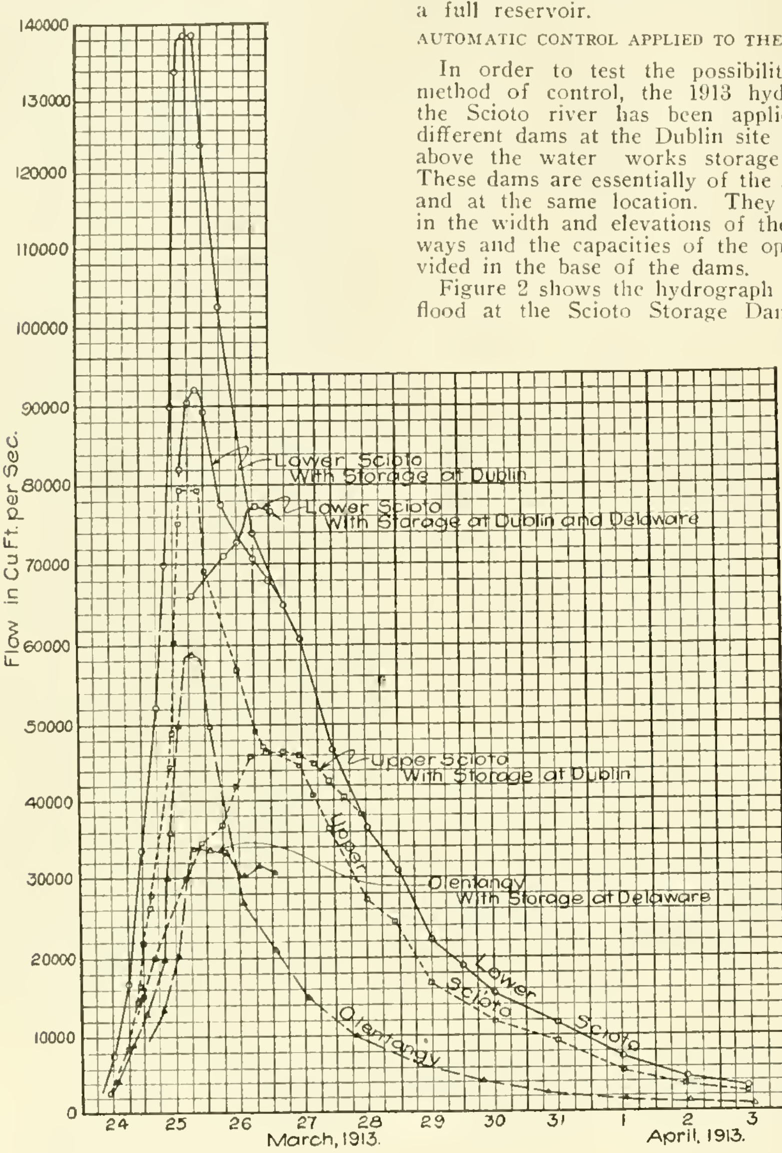


Fig. 3. Estimated Hydrographs at Columbus and the Effect of Reservoirs, Flood of 1913.

AUTOMATIC STORAGE.

As has been previously stated, where retention recsrvoirs are so seldom used as must be the case in protecting Columbus against the great flood, it is very important that as little dependence be placed upon human agency in the manipulation of the flood retention reservoirs as possible. It is desirable that so far as possible, they be made automatic in their operation. It is belived that a plan can be worked out whereby ordinary floods may be passed through the base of the dam through proper openings at a comparatively small head, thus utilizing only the bottom portion of the retention reservoir where there is the least volume per unit of height, leaving the major portion of the depth for the storage of greater floods. The outlet openings in the dam can be so proportioned that the greatest flood to be expected will pass through the openings provided, supplemented possibly by crest spillways at the head due to a full reservoir.

AUTOMATIC CONTROL APPLIED TO THE 1913 FLOOD.

In order to test the possibilities of this method of control, the 1913 hydrograph of the Scioto river has been applied to four different dams at the Dublin site immediately above the water works storage reservoir. These dams are essentially of the same height and at the same location. They differ only in the width and elevations of the free spillways and the capacities of the openings provided in the base of the dams.

Figure 2 shows the hydrograph of the 1913 flood at the Scioto Storage Dam, and this hydrograph has been used as a curve of inflow to the proposed storage reservoir near Dublin. This curve is marked No. 1. The curve marked No. 2 represents the outflow from the dam designated on the exhibit, having water passage in the base sufficient to pass a 20,000 second ft. flow at 23 ft. net hand. The free spillway is 90 ft. above the stream bed and 2,100 ft. in length. It will be observed that this dam reduces the maximum flow rate only a small amount, namely from 80,000 to 72,000 sec. ft. This is accounted for principally by the width of the free spillway and the storage volume at the spillway level. The total impounded storage at the crest level is 45,000 acre feet, and at the apex of the flood with 2.8 ft. head on the free spillway, the gross storage is 50,000 acre feet.

Curve 3 represents the result of increasing the spillway level 10 ft., its length remaining the same. Impounding 68,000 acre feet at the free spillway crest and 73,000 acre feet at the apex of the flood, this dam would be effective in reducing the maximum flood flow from 80,000 ft. observed during the flood to 58,000 second feet. Curve No. 4 represents the conditions that would be obtained at the dam last described, except that the spillway is reduced to 130 ft. in width, which necessitates the water rising at the apex of the flood to a depth of 7 ft. above the spillway crest, at which level the gross storage is 86,000 acre feet. Under these conditions the inflow of 80,000 second feet would be reduced to an outflow of 47,000 second feet. The curve as drawn traces the outflow to the end of the flood.

Curve No. 5 represents the result of applying the 1913 flood to the dam last described, except that the area of the openings in the base of the dam is slightly increased, permitting 22,000 sec. ft. to pass through, under a net head of 15.5 ft. This results in an outflow curve substantially the same as No. 4 in maximum rate of outflow. It differs from No. 4 principally in that the higher rates of outflow are of a little longer duration and the stored water will run out more rapidly after the apex of the flood.

TIME OF OCCURRENCE.

The maximum flows of the Olentangy and Scioto Rivers occurred at Columbus at very nearly the same time, and in the design of channels which must be based on maximum flow, the hour of its occurrence is not important. In the consideration of storage reservoirs, however, it becomes of importance to fix the hour that various flow rates are reached at Columbus, for the flows can only be controlled at the storage reservoirs and the effect at Columbus can be observed only by the addition of simultaneous flow rates that will prevail in the Scioto and Olentangy rivers at Columbus.

As bearing upon this matter, Prof. J. Warren Smith at the Columbus Weather Bureau necessarily observes these matters in preparing his flood forecasts. From a study of floods previous to that of 1913, he has observed that the flood apex at Mound St. usually occurs from four to six hours after the apex at the Scioto Storage Dam 7.9 miles upstream. This corresponds to a rate of travel for the flood wave of 1½ to 2 miles per hour.

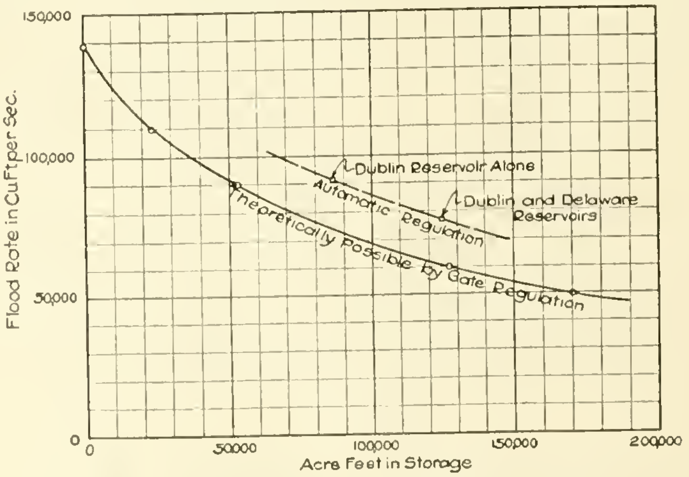


Fig. 4. Effect of Various Storage Volumes in the Reduction of Flood Rates, Columbus Flood of March, 1913.

The 1913 flood wave travel between these gages was not easily estimated because the levees between these gages were overtopped befor the apex of the flood at the Storage Dam, and the highest reading at Mound St. preceded the highest reading at the dam. Some light on this point can however be obtained by observing the rate of wave travel between the dam and the Sewage Works' gage.

Although the water at the dam reached its maximum about 2:30 p. m., it was nearly as high at 11 a. m., and the latter hour marks the end of the very rapid rise. The Sewage Work's gage reached its maximum water level at 5 p. m. The difference in time is six hours, the distance 11.4 miles, and the rate of travel 1.9 miles per hour. This corresponds quite closely to the observations of Prof. Smith on former floods.

The mouth of the Olentangy is 1.5 miles above the Mound St. gage, but the maximum should reach Mound St. not much later than the mouth of Olentangy, because all the bottom lands which tend to reduce the travel of the flood wave lie above the Olentangy mouth. It is believed under these circumstances, that it is fair to assume that under the late flood conditions the maximum at the Storage Dam should be effective at the mouth of the Olentangy about four hours later.

WAVE TRAVEL ON OLENTANGY.

Prof. Smith has likewise observed that maximums at Delaware appear at Columbus from six to twelve hours later. The distance between these places by river is 28 miles and the rate of travel is 2.3 to 4.7 miles per hour. The flood wave travel in the 1913 flood was indeterminate by the direct observation of river heights, as no gage is maintained immediately above Columbus, and in the city the gage heights are the resultant of the sum of the flows of the two rivers.

Some light is thrown on the matter by the fact that the levees above Mound St. were not overtopped until between 9:00 and 10:00 a. m., up to which time the channel below the mouth of Olentangy was taking all the water brought down. This channel, as has been previously estimated, was capable of taking normally about 50,000 sec. ft., and may have taken under the special flood conditions as high as 70,000 sec. ft. The flood apex at Delaware occurred about 11:30 a. m., and if we set the Delaware hydrograph about eight hours forward and similarly set the Scioto hydrograph four hours forward, the sum of the hydrographs will indicate about 65,000 sec. ft. of flow total at the junction of the rivers, and this is very nearly in accordance with all the facts bearing upon the matter that we can determine. The upper curve on Fig. 3 represents the combined hydrograph of the two rivers at their mouths estimated upon this basis from the observations at the Scioto Storage Dam and the Delaware bridge.

EFFECT OF AUTOMATIC STORAGE.

Upon Fig. 3 there has also been platted the outflow curve from the Dublin storage reservoir corrected to apply to the mouth of the Upper Scioto. The diagram also shows the estimated flow of the Upper Scioto without storage, applied to the same place and the same facts regarding the Olentangy River with and without the Delaware storage.

In order to observe the effect of automatic storage at Dublin alone, the storage curve for the Scioto has been added to the flow of the Olentangy without storage. This would result in a total flow at the river mouths, as shown by the curve marked "Lower Scioto with Storage at Dublin." This storage would have the effect of reducing the maximum flow rate at Columbus from 139,000 sec. ft. to 92,000 sec. ft., the maximum occurring at about 9:00 p. m. under the 1913 flood conditions.

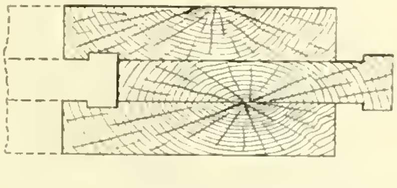


Fig. 1. Interlocking Wood Sheet Piling.

With storage reservoirs at Dublin and at Delaware on both rivers, the maximum flow at Columbus near the apex would be as shown by the curve marked "Lower Scioto with Storage at Dublin and Delaware" and would reach a maximum of about 77,000 sec. ft. at about 7:00 p. m. of the day following the flood apex without storage.

THEORETICAL STORAGE.

As has been previously stated, it is theoretically possible by the manipulation of gates of great capacity to so control the flood that the flow is allowed to rise to a predetermined figure, at which point the gates would be slowly closed to such per cent of opening as would maintain the flood at the desired amount within the limits of storage in the reservoirs. The volume in storage required is the gross amount of water passing during a flood over and above the predetermined amount to which it is desired to regulate the outflow from the storage reservoirs. The degree of regulation thus effected may be regarded as the unattainable toward which the automatic storage should tend, and it will therefore be useful to ascertain how closely automatic storage approaches to the gate regulated storage in its results.

Figure 4 shows the amount of storage required to reduce the flood apex at Columbus by various amounts during the flood of March, 1913. Upon Fig. 4 is also shown the result of introducing automatic storage at Dublin, and also by the two reservoirs at Dublin and Delaware. It is thus indicated that from 50 to 60 per cent more storage is needed with reservoir control by automatic outlets than would be theoretically possible by gate control, or stated in another way, the automatic storage is from 75 to 80 per cent as efficient in reducing flood rates as the gate regulated storage.

Practically, the efficiency of the automatic storage is greater than the above figures would indicate, even assuming that it would be practicable to install gates that would operate satisfactorily and pass large flow volumes under both high and low heads. Some considerable portion of the lower part of the reservoir (the reservoir being in the valley of the main stream) would necessarily be occupied by the flood before the gates are closed, particularly if, as would probably be expedient, the large flows pass through the gates at considerable head.

In a flood water storage project involving a large number of storage reservoirs upon the headwaters of a stream, the flow rates passing the gates would be comparatively small, and probably the gates would be operated under low head. Under such circumstances gate regulated storage is doubtless quite practicable so far as the operation of the gates is concerned. In the regulation of the tremendous flows, however, that must prevail immediately above Columbus, and the high heads under which the gates must operate, the practicability of gate regulated storage is in our opinion questionable. To a limited extent it may be utilized in improving the conditions of the automatic storage in an emergency.

A New Interlock Wood Sheet Pile.

(Staff Article.)

A new form of wood sheet piling which is being used on work of the Mississippi River Commission is shown by the accompanying drawing. The new pile is so simple that the drawing alone explains all the essential structural features. Due to the interlocking feature, this piling may be driven on a batter as well as vertical and will retain a tight joint underground. As furnished the pieces properly cut are shipped separately and then are spiked together to form "piles" as the work requires. Long leaf pine lumber is employed, and is cut to any length and size required. The control of this interlocking piling is held by the Alexandria Lumber Co., Ltd., Alexandria, La. For recent levee work the price bid on material for this piling on board barge at New Orleans, La., was $26 per M. ft. B. M.

Sewerage

Some Suggested Designs for Sewage Treatment Plants for Residences and Small Institutions.

Five sewage treatment plants suitable for use in connection with residences and small institutions are described in the following article; three of the designs are illustrated. Other articles bearing upon the design of sewage treatment plants for residences were published in ENGINEERING AND CONTRACTING of Feb. 8 and Oct 18, 1911, and Aug. 7, 1912.

The information given in this article is taken from a paper by Mr. Paul Hansen, Engineer of the Illinois State Water Survey, before the recent annual meeting of the American Society of Agricultural Engineers. The article gives data on the cost of plumbing a house, on the design and operation of simple septic tanks, of a modified Imhoff tank adapted to the use of a household of ten persons, of a subsoil irrigation system, an intermittent sand filtration plant to handle the sewage from 25 persons, and a double contact filtration plant.

Cost of Plumbing.—An approximate estimate of the cost of plumbing a house is given in the following tabulation, the figures given relating to a two-story house with basement:

Soil pipe with necessary Y and T connections	$15.00	to	$30.00
Two-set tubs for laundry	20.00	to	60.00
Kitchen sink with drain board	7.50	to	30.00
Bath tub	20.00	to	200.00
Wash basin for bath room	7.50	to	10.00
Closet	17.50	to	75.00
Necessary piping and faucets for hot and cold water, including hot water boiler	20.00	to	40.00

Looking at the matter in another way, complete house plumbing including bath tub, sink and laundry tubs of wood and zinc, wash basin of enameled iron, water closet of porcelain may be installed for $200. Complete house plumbing including closet, wash basin and sink of porcelain and bath tub and laundry tub of enameled iron may be installed for $300. The first represents cheap and somewhat flimsy construction; the latter first class and substantial construction.

Simple Septic Tank.—The simplest form of treatment device that may be used for a single house comprises merely a tank, capable of holding the sewage produced in a period of 24 hours or longer. Mr. Hansen suggests a tank of the following dimensions, to cost approximately $30 with concrete roof and floor and brick walls: Inside plan, 2x8 ft.; depth of tank, 6 ft.; depth of sewage, 3 ft. 10 ins.; simple wooden baffle, 1 ft. 6 ins. from inlet, extending 12 ins. into sewage; lower end of 6-in. outlet tile, a 90° bend, submerged 18 ins.; the tank bottom slopes to the center, and the sludge is withdrawn from a sump through an outlet by opening a shear valve. The costs given in this article are relative figures, especially applicable to average Illinois conditions.

The effluent from a tank of this character contains no coarse solids, is likely to be dark in appearance and upon close observation will be found to carry finely divided suspended particles. It may be discharged into a perennial stream of moderate size without producing objectionable conditions, but where no such stream is available some other method must be utilized. In a sandy or gravelly soil the liquid may best be disposed of by permitting it to flow into a system of open jointed tile drains placed at 1 ft. to 18 ins. below the surface of the ground. The tile drain may be made up of a 4-in. farm drain tile and should have a length of about 100 ft. for every member of the household. The character of soil may materially vary the amount of tile necessary. To get the best results the liquid should be applied to the tiles in the form of a dose discharged rapidly from a syphon within a few minutes once every 12 or 24 hours. This can be accomplished by building in conjunction with the tank a small dosing chamber equipped with an automatic discharge syphon. Syphons are subject to derangements and may be omitted in favor of a more liberal use of tile. An arrangement for a sub-surface system of tile drains following passage through a septic tank is suggested by Mr. Hansen as follows:

Septic Tank, Dosing Chamber and Subsoil Irrigation.—The septic tank is 2x6 ft. in plan and carries a depth of sewage of 4 ft. Its details are similar to those described above. The dosing chamber immediately adjoins the septic tank and is provided with an automatic siphon. After leaving the siphon the effluent passes into a system of drain tiles consisting of distribution and collector tiles, the former being from 12 to 18 ins. below the surface and the latter 1 ft. lower still and 5 ft. to one side of the distributor. The collector pipes may generally be omitted. Where the soil is of a finely divided character and capable of absorbing large quantities of moisture as is characteristic of most of the glacial drift the same system may be utilized, but can be relied upon adequately to take care of the sewage only during periods of small rainfall. This, however, often proves to be all that is needed for in wet weather the sewage may be discharged directly into a stream which will then have flow enough to provide the dilution necessary for inoffensive oxidation. Sufficient dilution may even obtain within the drain tile itself. It thus sometimes happens that the discharge of the effluent into the upper end of an existing long tile drain accomplishes all that is required, for in dry seasons the sewage will soak into the ground, while in wet seasons it will be carried to the outlet of the drain along with a sufficient quantity of water to render the liquid inoffensive.

In certain forms of clayey soil any attempt to cause the sewage to soak into the ground will prove futile, but this difficulty may be overcome by artificially placing beds of sandy or gravelly material about the tile. When this is done it is necessary to place sub-drains for carrying off the more or less purified liquid after it has passed through sand or gravel. This arrangement constitutes the principle of a well-known proprietary device for sewage treatment and can be counted upon to produce satisfactory results as long as a sufficient quantity of porous material is used. For large installations, however, it would prove costly and uneconomical.

It is difficult to lay down any hard and fast rules relative to the design and arrangement of subsurface sewage disposal systems. The proper length of drain tile, the necessity of collector drains and the desirability of surrounding the drains with porous material must under many conditions be determined by the method of cut and try. Where the system is applicable, however, it is certainly the simplest, least offensive and easiest to maintain.

Modified Imhoff Tank for Residential Service.—With certain forms of secondary sewage treatment devices, and often where sewage is to be discharged into a watercourse it is desirable to maintain the sewage in as fresh a condition as possible; that is to say, it is desirable to maintain the oxygen content of the sewage. To accomplish this Mr. Hansen suggests a modification of the Imhoff tank as shown in Fig. 1 herewith. As will be noted the tank utilizes a small compartment for sedimentation purposes and a lower compartment for the digestion of the solid matters. The solid matters which are settled out in the upper compartments gain access to the lower compartments through a narrow slot so trapped that the liquids in the two compartments may not readily intermingle. The sedimentation chamber must be made very much larger than is customary in connection with municipal sewage treatment works for the reason that the very uneven rate of sewage flow from an individual residence would subject the sedimentation compartment to violent disturbances. For this reason an average period of 5 or 6 hours sedimentation should be provided.

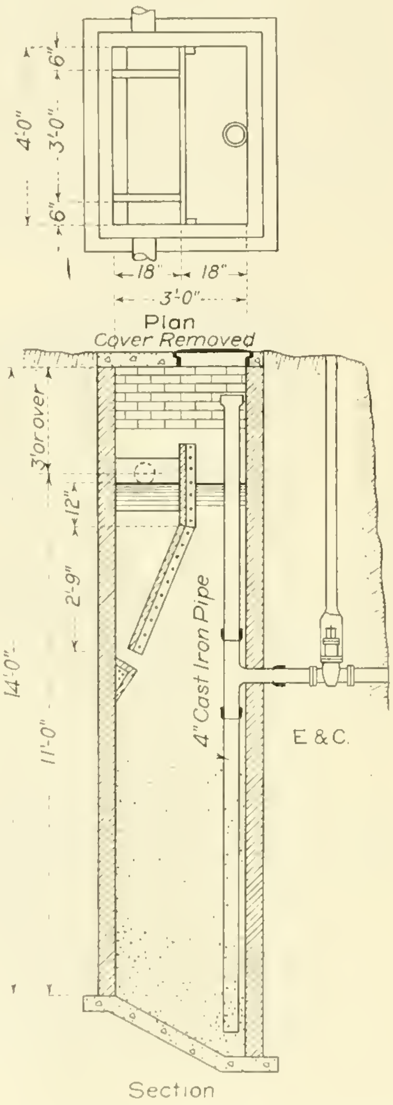


Fig. 1. Suggested Design of Modified Imhoff Sewage Tank for Household of Ten Persons.

This tank has an advantage in that the sludge or solid matter is thoroughly digested and may periodically be discharged into some nearby depression without giving rise to objectionable odors. When dry it has the consistency of rich loamy earth and may advantageously be spread on lawns and gardens. The modification of the Imhoff tank here shown is adapted to the use of a household of ten persons. The cost of such a tank would be about $60.

Intermittent Sand Filtration.—On large country estates or for isolated institution such as alms houses, asylums, sanatoria, etc., perhaps the most economical and highly efficient system is so-called intermittent sand filtration. Figure 2 shows an intermittent sand filtration plant that will adequately care for the sewage of about 25 persons (generally they should not be built for populations of less than 25). The plant comprises also an Imhoff tank and a dosing chamber. The filters consist of a bed of sand 3 ft. in thickness, resting upon a carefully constructed under drain system of drain tile and a graded gravel. The total area of the installation is figured on the basis of one acre for every 750 persons tributary to the sewers. For a household of 25 persons there would be required 0.033 acres, or about 1,500 sq. ft. The sewage is applied to the filters intermittently and this is accomplished by a dosing chamber which is of such capacity that a single dose will cover one filter bed to a depth of 2 ins. The filter area should be divided into at least two beds, but preferably three. The latter arrangement permits one bed to be out of service for cleaning or repairs or for a protracted resting period without serious reduction in filter area and without interfering with the proper operation of other beds. The automatic dosing apparatus is so arranged that it will automatically discharge upon first one bed and the other and valve arrangements are such that any two beds may be maintained in service.

Intermittent sand filters will not operate without attention, and they should be visited at least once every few days to see that all parts are in proper working order and to keep the sand beds free from weeds. At greater intervals, perhaps once per month, it becomes necessary to take or scrape the sand lightly so as to break up the surface of the mat formed by the solid matter in the sewage, but with proper care this process should result in but very slight loss of sand. During the winter in cold climates it is necessary to furrow or mound sand beds in such way as to cause the formation of a sheet of ice resting upon the top of the furrows. This will effectively prevent the freezing of the sewage as it passes along the troughs of the furrows underneath. The automatic dosing apparatus requires attention as already noted, but with a well settled sewage and siphons of reliable make derangements will be infrequent.

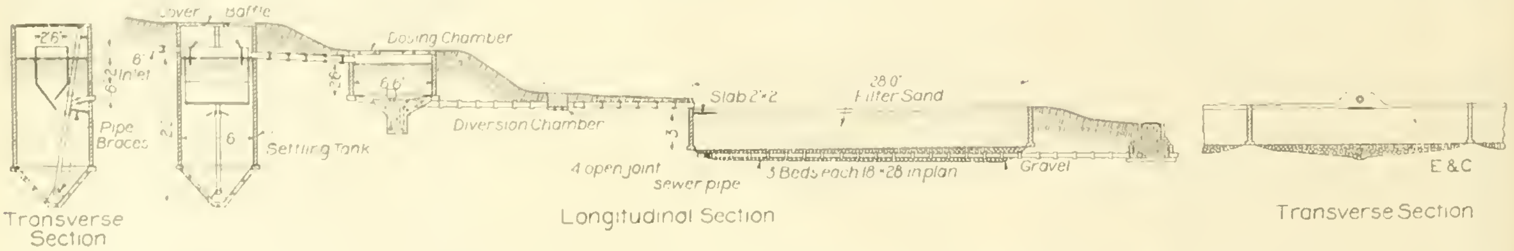


Fig. 2. Sectional Elevations of a Suggested Intermittent Sand Filtration Plant, With Preliminary Sedimentation, for Treating Sewage From Household or Institution of 25 Persons.

Double Contact Beds.—A somewhat more extensive method of sewage treatment and one which is especially adaptable to very large farm houses, country clubs and institutions and for use in locations where the production of odors is not permissible is treatment in so-called contact beds. An installation comprising double contact beds is shown in Fig. 3. It is assumed that the contact treatment will be preceded by sedimentation. The contact beds are water tight basins filled with gravel, broken stone, hard burned cinders, broken brick or any other material which presents a fairly rough surface and which is not easily disintegrated. The size of the material should not average less than ½ in. in diameter or be greater than 1½ ins. in diameter. Means are provided for permitting the sewage to flow into this bed and to fill the interstices of the material which it contains where it may remain in contact for a definite period of time. The bed is emptied and the period of contact is regulated by means of a time controlled siphon. In the design as shown in Fig. 3 the effluent from the first contact bed passes into another contact bed of exactly similar construction. Upon emerging from the second contact bed the liquid will be fairly clear in appearance and will have no offensive odor, thus rendering it suitable for discharge into a small watercourse. For the purpose of minimizing odors a specially constructed inlet arrangement to the bed is provided which prevents the sewage from being exposed to the atmosphere. The device consists simply of short half tile 14 to 18 ins. in diameter and laid with large open joints. Within the invert of this half tile is placed fine broken stone or gravel and over the top is placed a readily removable roof. The device also has another advantage in that it localizes all clogging in the fine material within the half tile. When this becomes clogged so that it will not readily permit the entrance of the sewage into the bed it may be easily removed and replaced with new material.

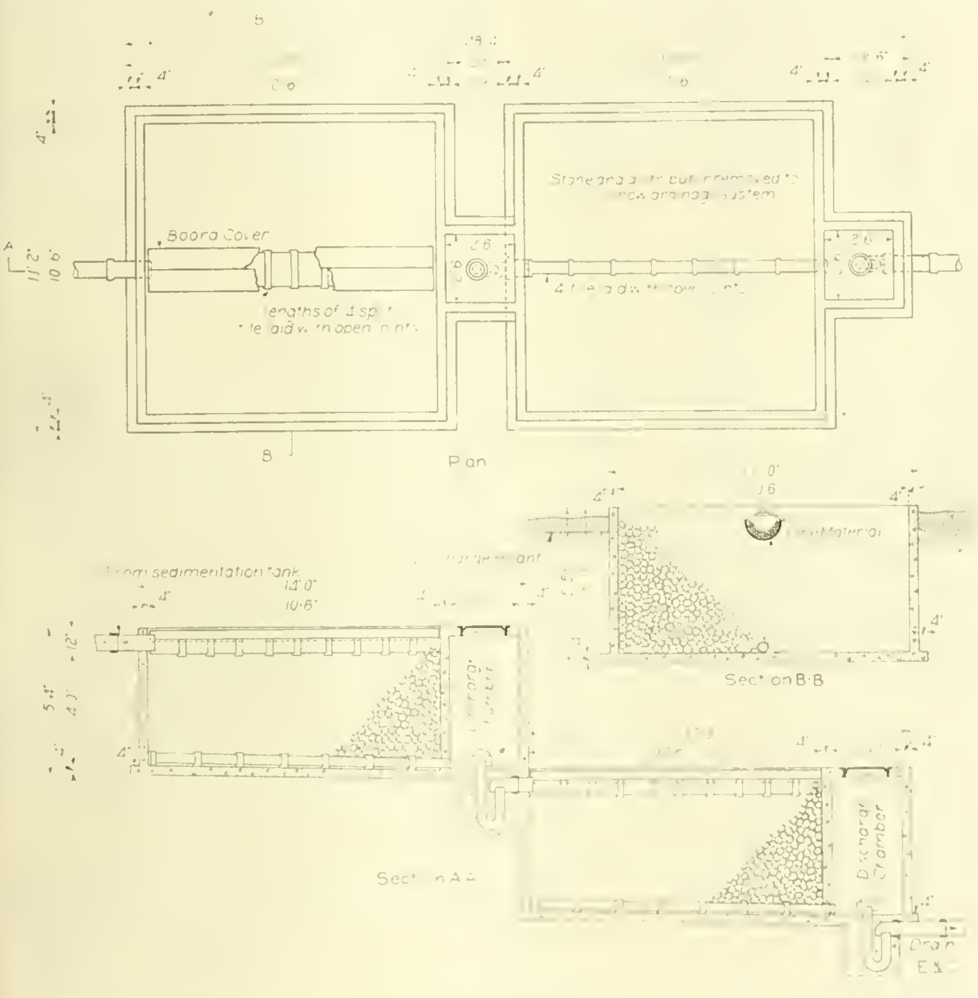


Fig. 3. Suggested Design for Double Contact Sewage Treatment Plant for Large Residences, Country Clubs or Small Institutions.

The Design and Operation of Small Sewage Pumping Stations.

The design and operation of sewage pumping stations is a subject upon which very little has been written, and least of all on the design and operation of small pumping stations. One reason, perhaps, is that the use of sewage pumping stations is avoided as far as possible in the design of sewerage systems for the smaller towns and villages, even at the risk of flattening grades to a point where thorough scouring may not always occur at low flows. However, on small areas and frequently on larger areas where conditions are imperative, pumping stations are built to lift the sewage into the outfall sewers or to works for treatment, as required.

Several types of designs for sewage pumping stations were described and illustrated in Engineering and Contracting of Jan. 29, 1913. The results of a study made to determine the most economical motive power for pumping sewage at Lynn, Mass., were given in tables and diagrams published in this journal of Jan. 7, 1911. An important contribution to the literature on this general subject, with special reference to small sewage pumping stations was made by Messrs. Samuel A. Greeley and Langdon Pearse in their recent paper before the Illinois Society of Engineers and Surveyors. The paper follows.

In the design of the small stations, there are several points to be considered. (1) The preparatory treatment by screens required to protect the pumps. This is more important on small installations, particularly those where attendance is intermittent. Bar screens with net openings of ½ to ¾ in. are common. There should be ample provision for ready cleaning and accessibility. (2) Provision of storage in a suitable well or tank, in order that pumps of reasonable size may be used with intermittent pumping. Sometimes pumping installations may serve to dose treatment works. The design should provide bottom slopes steep enough to keep the sludge moving to the pumps. (3) The question of location and character of buildings. (4) The total cost for operating and maintenance capitalized and added to the construction cost will give the means for comparing the relative merits of different sources of power available.

For the small sewage pumping station, where intermittent attendance is planned, the use of electrically driven pumps would seem to be ideal, in that they are clean, self-contained, noiseless, and easily controlled by well developed devices for automatic starting and stopping. Attendance is thereby reduced to a minimum. The equipment is usually compact and may be installed in comparatively small space. The cost of pumping depends on the rate at which power is sold in the territory and is clouded only by the necessity that pumping usually is somewhat intermittent, rather than continuous. This increases the peak load.

The design of electrically driven pumps, direct-connected, has been worked out to a point where they operate efficiently and economically. The recognized points of design would seem to be the use of vertical motors with a suitable oil bearing to support the moving parts of the motor, the support of the weight of the moving shaft and runner of the pump on a ball or roller bearing, placed above high water where it can be readily inspected, adjusted and oiled, and a connection with the motor of the same with a flexible coupling. Steady bearings are desirable on the shaft at frequent intervals with provision for oiling and accessibility. It is highly desirable with any type of pump installation for sewage that the same be set in a dry well, so that none of the moving parts requiring lubrication will be exposed to the sewage, which may frequently contain grit, which is destructive to bearings. The necessity of keeping gritty water out of water seals and bearings has not been always recognized by designers, as operating men have found.

The use of gas engines has also been developed, operating on natural gas or artificial gas from the mains, but with a very few exceptions—notably, a small pumping station in Newton, Mass., handling sewage of about 760 people, with two 3½ in. centrifugal pumps—the developments have been largely in the larger stations of the size of those at Columbus, known as the East Side pumping station, where two 60 HP. gas engines operate two 10-inch centrifugal pumps, chain-belt driven; and at Hegewisch, in Chicago, where the pumps are run on gas from the city mains. The latter installation is belt-driven and pumps the entire sewage, as well as a large proportion of the storm flow; whereas, the Columbus station simply pumps the dry weather flow and a very small proportion of the storm sewage. The Newton station is semi-automatic, being started two or three times a day by a fireman from the neighboring water works pumping station. A stopping device is installed. The use of producer gas for the purpose of sewage pumping, as well as water works pumping, appears attractive, but has not been installed to any great extent for small installations, since the attention of the promoters of producer gas has been directed largely to industrial installations, and the municipal field has been left untouched. There is, however, a station operating at Pennypack Creek in Philadelphia.

Gasoline and kerosene for sewage pumping are, of course, available, but with the present price of gasoline, the costs are high. Kerosene is less expensive, but seldom used. However, engines working on distillate, as is common in California, should prove successful for sewage pumping as they have done for irrigation purposes. Madison, Wisconsin, pumps all the dry weather flow with engines converted to run on kerosene. Sewage installations, however, require to be operated intermittently, as a rule, and therefore need automatic control where attendants are lacking. This favors the direct-connected, compact electrical units. The gas engine and other types of explosion engine require somewhat more attention and care in order to secure successful operation. For the small sewage pumping station in the small town, steam driven machinery is generally out of the question, as the efficiency is low and the cost of attendance high for such small installations. It is only when the installations are operated in connection with other works that steam appears to be attractive for small stations, working on low lifts and handling amounts of sewage under 1,000,000 gals. per day.

Some interesting data on the cost of small plants, collected by us, are given in Table 1, showing the cost calculated to water horsepower, regardless of the efficiency produced. We have also tabulated the costs of operation as taken from the various records in print. These costs of operation are given with the reservation that if used they should be carefully studied from the standpoint of actual application to a case in hand, as there is a tendency in many municipal records as given out to make as good a showing as possible by giving only the charges apparently chargeable to the pumping account and carrying other incidentals where necessary in other ways.

TABLE I.—COST DATA AND STATISTICS ON SEWAGE PUMPING.

Station—	Type.	Approximate Capacity.	W.HP. Output.	Cost of construction per W.HP. capacity.	Cost of operation per W.HP. output per year— Labor.	Fuel.	Repairs, supplies, etc.	Total.	Date of data.
1. Pennypack Creek	Producer gas	45.0	9.0	$ 450.00	$ 735.00	....	....	....	1913
2. Hegewisch	City gas	30.5	21.0	465.00	1,890.00	$ 210.00	$ 323.00	$2,423.00	1912
3. Alewife Brook	Steam	50.0	7.7	....	828.00	210.00	95.00	1,133.00	1912
4. Quincy	Steam	70.0	15.2	....	417.00	110.00	48.00	575.00	1912
5. Saratoga	Electric	31.8	....	170.00	....	....	....	....	1907
6. Hudson	Electric	8.0	....	450.00	....	....	....	....	1907
7. Newton, Mass.	Electric	2.5	0.14	1,840.00	775.00	1,020.00	1,105.00	2,900.00	1906
8. Plainfield—									
Monroe Ave.	Air	....	0.25	....	....	760.00	....	....	1912
Randolph Rd.	Air	4.5	1.00	1,040.00	....	586.00	....	....	1912

NOTE—A water horsepower is figured from the rated capacity or actual output of water per day and the total lift.

NOTES TO ACCOMPANY TABLE I.

(1) Pennypack Creek Sewage Disposal Works, Philadelphia, Pa. Present average quantity pumped, 1,000,000 gals. per 24 hours, against a 50-ft. head. Equipment consists of one 2,000,000-gal. and one 3,000,000-gal. vertical submerged centrifugal pump geared to gas engines; and one 150 HP. gas producer operated with anthracite coal.

(2) Hegewisch Sewage Pumping Station, Chicago, Ill. Built in 1909. Present average quantity pumped, 1,500,000 gals. per 24 hours, against a 10-ft. head. Equipment, two 10,000,000-gal. centrifugal pumps, and two 40 HP. gas engines. City illuminating gas supplies the engines.

(3) Alewife Brook Pumping Station, Metropolitan Sewerage Works, Boston, Mass. Present average quantity pumped, 3,500,000 gals. per 24 hours, against a 13-ft. lift. Equipment is two 4,500,000-gal. vertical centrifugal pumps, and one 13,000,000-gal. horizontal centrifugal pump; all operated by compound steam engines.

(4) Quincy Sewage Pumping Station, Metropolitan Sewerage Works, Boston, Mass. Present average quantity pumped, 4,000,000 gals. per 24 hours, against a 22-ft. lift. The equipment is one 3,000,000-gal., one 5,000,000-gal. and one 10,000,000-gal. centrifugal pump, operated by compound condensing engines.

(5) Saratoga Springs, N. Y. Daily pumpage varies from 1,500,000 to 3,000,000 gals. per day, against a 30-ft. lift. The equipment is three vertical submerged centrifugal pumps with capacities of 1,500 gals. per minute each and three 20 HP. induction motors.

(6) Hudson, Mass. The equipment consists of two vertical centrifugal pumps, each with a capacity of 500 gallons per minute against a total head of 35 ft. Each pump is operated by a 15 HP. induction motor.

(7) Newton, Mass. Average quantity pumped 1906; 25,565 gals. per day against a 30-ft. lift. The equipment consists of two 3½-in. vertical submerged centrifugal pumps geared to 5 HP. vertical gasoline engines.

(8) Plainfield, N. J. Randolph Road Pumping Station. Present average quantity pumped, 130,000 gals. per day against a 40-ft. lift. The equipment is as follows:

Two 10-in. by 6-in. Ingersoll-Rand compressors.
Two 7.5-HP. electric motors.
One gasoline engine for reserve.
One 42-in. by 8-ft. air receiver.
Two duplex displacement ejectors.

The plant capacity is approximately 500 gals. per minute.

Monroe Ave. Pumping Station. Present average quantity pumped is approximately 30,000 gals. per day against a 40-ft. lift. The equipment is as follows:

Two Blaisdell air compressors.
Two 5-HP. electric motors.
Two Blaisdell 150-gal. ejectors.

The plant capacity is approximately 300 gals. per minute.

Results of Studies Made to Determine Effect Upon Fish Life of Sewage and Sewage Treatment Plant Effluents.

The effect upon fish life of sewage and the effluents from sewage filters has received little investigation in this country, although in a general way it is known that owing to increased pollution fish have disappeared from many streams in which they once were abundant. The effect of sewage pollution upon shellfish, and the areas from which shellfish are taken has been investigated during the past 10 or 15 years chiefly because shellfish are likely to be the bearers of infectious diseases. An important article on shellfish conservation and sewage disposal was published in this journal for Oct. 29, 1913.

Important studies were made by H. W. Clark and George O. Adams, at the Lawrence experiment station of the Massachusetts State Board of Health in 1912, to determine the effect upon fish life of sewage and effluents from sewage treatment plants. Observations were made of the effects upon fish life of crude sewage, diluted sewage, effluents from sand contact and sprinkling filters, both undiluted and diluted. The effects of aerated and non-aerated effluents upon fish also were observed. Observations also were made upon the amount of oxygen consumed by fish life, upon the solution and diffusion of oxygen in water, and upon the liberation of oxygen by green, growing water plants. The matter following is taken from the report of these studies as published in the annual report of the Board for 1912:

From a practical point of view the question of fish life in streams and lakes in a settled community is one of the value of such life in a certain stream or lake compared with the cost of maintaining such freedom from pollution or such restrictions in the use of streams as will allow the continuance of this life. It goes without saying that looked at in this way the cost of preserving a stream in a condition that allowed fish life might be exceedingly large as compared with the value of the fish saved. This would be especially true in manufacturing districts, as in such places the streams are chiefly valuable for power and the carrying away of the waste products of the district more or less purified. Treating these wastes sufficiently to prevent the stream from becoming a nuisance, moreover, may not keep it sufficiently pure to encourage the life of fish within it, and many streams seem fated to become sources of power and the carriers of wastes, and find their chief value when so used. Practically all that can be expected in many instances is that the sewage or wastes entering such streams shall at all times of the year be so purified before entrance or so diluted by the flow of water that the stream will not be a nuisance to the community through which it flows, but rather a source of enjoyment to such community.

From a scientific standpoint there are a number of questions to be considered in this study of pollution and fish life, the most important being as follows: (1) the amount of pollution that a stream may receive without decreasing the dissolved oxygen of the water to such a degree that fish can no longer live in the water: (2) the effect of the various constituents of sewage and trade wastes upon fish, and the effect of these various constituents upon fish even when a plentiful supply of dissolved oxygen remains in the water after receiving these wastes; (3) the amount of oxygen consumed by the fish themselves; and (4) the rate of absorption of oxygen by polluted waters.

There are other questions involved regarding which a series of investigations was carried on during 1912 at the experiment station, a portion of them being described here. The experiments were made largely in 5-gal. glass aquaria, and the commonest fish of the region —shiners, suckers, chub and carp—were used. Most of the carp and shiners have weighed from 1 to 2 ounces each, while the suckers have weighed from ¼ to ½ lb.

EFFECT OF SEWAGE.

In the first experiments the fish were immersed in undiluted Lawrence sewage, well aerated, and containing at the time of immersion from 50 to 100 per cent of saturation of dissolved oxygen. Notwithstanding the presence of this dissolved oxygen, all the fish in these experiments apparently became crazed when immersed and died almost immediately: that is, within a few minutes after immersion.

EFFECT OF EFFLUENTS FROM SEWAGE FILTERS.

Following this, experiments were made with the effluents from sand, contact and trickling filters and with mixtures of sewage and water, contact filter effluents and water and trickling filter effluents and water. In this series of experiments a supply of dissolved oxygen was maintained when necessary by blowing air through the liquid used. The results were as follows:

(a) It was found that when water and sewage of the strength used were mixed in equal portions, and this mixture kept aerated, fish life could exist almost indefinitely, but with a larger proportion of sewage the fish died within a few minutes or, at most, in an hour or two after their immersion. Even in the equal mixtures of sewage and water, a number of fish died quickly, and only the stronger individuals survived, but such as did survive were kept in this mixture alive and well for 14 days, and apparently could have continued to live for an indefinite period. The sewage used was freed from matters in suspension before being mixed with the water.

(b) It was found that aerated effluents from contact and trickling filters in successful operation supported fish life for 14 days without any apparent discomfort to the fish immersed in these effluents.

Table I presents the analyses of the mixture of sewage and water, and of the effluents from contact and trickling filters used in these experiments.

EFFECT OF SEWAGE AND EFFLUENTS WHEN NON-AERATED.

Experiments were next made to determine the volume of water (tap water) necessary to mix with sewage and the effluents from trickling filters in order that fish life might be sustained under what might be called stagnant conditions; that is, sustained without the introduction of dissolved oxygen by aeration as in the previous experiments. After several trials it was found that but 10 per cent of sewage could be mixed with 90 per cent of water and the fish survive in good condition for 14 days. Increasing the sewage above this percentage invariably caused the exhaustion of the oxygen and the death of the fish within a few hours. It was found that only 25 per cent of water was needed in 75 per cent of the best trickling filter effluents in order that fish might live under these stagnant conditions for 14 days, this mixture always containing enough oxygen to support life, and the effluent present having apparently no harmful effect upon the fish. With the contact filter effluents equal mixtures were necessary, that is, 50 per cent of the effluent to 50 per cent of tap water.

TABLE I.—ANALYSIS OF FILTER EFFLUENTS AND DILUTE SEWAGE USED IN EXPERIMENTS.
(Parts per 100,000.)

	Contact filter effluent.		Trickling water effluent.		Equal mixture of water and sewage.	
	At start.	After 14 days.	At start.	After 14 days.	At start.	After 14 days.
Free ammonia	1.5500	2.8000	1.8000	2.8800	1.9000	2.8000
Albuminoid ammonia—						
Total	0.2700	0.2880	0.2800	0.3480	0.2100	0.2280
In solution	0.1540	0.1840	0.1800	0.1220	0.2100	0.1480
Chlorine	11.3000	12.4000	10.7000	12.1000	7.3000	7.0000
Nitrogen as—						
Nitrates	0.0300	0.0400	2.0100	2.1200	0.0200	0.0200
Nitrites	0.0000	0.0000	0.0100	0.0600	0.0024	0.0180
Oxygen consumed	1.7800	1.8000	1.8200	2.2200	1.2400	1,4600
Solids in suspension—						
Total	......	......	......	......	0.0000	2.3000
Volatile	......	......	......	......	0.0000	2.2000

LIMIT OF DISSOLVED OXYGEN.

At the beginning of the last experiments described, that is, at the time of the immersion of the fish, the dissolved oxygen in the mixtures was 0.50 part per 100,000. During the experiment it became reduced in the water and effluent mixture to as low a point as 0.11 part per 100,000, and in the mixture of sewage and water, to 0.14 part per 100,000, but without any apparent discomfort to the fish in these mixtures.

The greatest consumption of oxygen by the organic matter present in such mixtures occurs in the first few hours, and it was found that if the initial supply of oxygen was great enough to last through this early period the oxygen would then be increased by absorption from the air.

In a third experiment of this nature, where a mixture of equal portions of tap water and trickling filter effluent was made, the dissolved oxygen at the start was 0.80 part per 100,000 and was not reduced below 0.10 part per 100,000 during the fourteen days of the experiment.

TABLE II.—ANALYSIS OF SEWAGE USED
(Parts per 100,000.)

Ammonia			Kjeldahl nitrogen			
	Albuminoid					
Free.	Total.	In solution.	Total.	In solution.	Chlorine	Oxygen consumed.
5.80	0.86	0.35	1.38	0.79	12.10	1.10

In further investigations, in regard to the amount of dissolved oxygen that would sustain fish life, the following three experiments were made: Sewage of the strength shown in Table II was so mixed with water that three dilutions containing, 5, 10 and 15 per cent, respectively, of sewage were obtained. In these dilutions the dissolved oxygen was at first 0.88, 1.00 and 0.85 part per 100,000. In each dilution fish were immersed and in each of the three dilutions they lived for one week without discomfort; that is, until the end of the experiment. The minimum amount of dissolved oxygen found during this period in each of these dilutions was 0.47, 0.28 and 0.15 part per 100,000, respectively.

EFFECT OF SAND FILTER EFFLUENTS.

In six experiments in which fish were immersed in well-purified, highly nitrified effluents from sand filters receiving sewage, the three fish immersed in each experiment died in from three hours to three days after immersion. All of these effluents but one were clear but slightly acid, this acidity being due to insufficient base to combine with the nitric acid present. One effluent was alkaline, but contained considerable matter in suspension, this matter being largely silica and alumina. When this suspended matter was removed the fish lived in this effluent without any apparent discomfort for 14 days or during the length of the experiment. Other acid effluents from sand filters when neutralized and filtered to remove precipitated $CaCO_3$ and $MgCO_3$, etc., supported fish life. The alkaline suspended matters at times separating from such effluents and from sewage, gathering at the outlet of the fish's gills and at the joints in their scales, gave the appearance of a fungous growth upon the fish.

Table III presents the results of analyses of the effluents used in these experiments.

CONSUMPTION OF OXYGEN BY FISH LIFE.

The large amount of dissolved oxygen breathed by fish is little realized. In studying this point, the following experiments were made; 13 small fish weighing altogether 200 grams were placed in a salt-mouth gallon bottle filled with water, and the dissolved oxygen determined at the beginning and at the end of the experiment.

In a second experiment four fish weighing 340 grams were used. In all, nine experiments of this kind were made. Averaging these experiments it was found, speaking generally, that all the oxygen held in solution in a gallon of water at 60° F.—practically 1 part per 100,000—would be consumed or breathed in one hour by a weight of fish equal to 1 lb. This would mean that 1,000 lbs. of fish life would in about 15 days exhaust the dissolved oxygen in a layer of water 1 ft. deep and an acre in area, saturated at 60° F., assuming, of course, that dissolved oxygen was not added by absorption from the air. The oxygen consumed in this way remains in the water as CO_2 and hence the amount of CO_2 in surface waters sustaining fish life must be influenced to a considerable extent by the fish present.

TABLE III.—EFFLUENTS FROM SAND FILTERS RECEIVING SEWAGE
(Parts per 100,000.)

	Ammonia			Nitrogen as			
Experiment Number—	Free.	Total albuminoid.	Chlorine.	Nitrates.	Nitrites.	Oxygen consumed.	Alkalinity.
1	0.1800	[illegible]	9.00	.21	.0021	.23	—0.9
2	0.2600	096[illegible]	7.30	1.61	.0032	.51	—0.3
3	0.9600	0960	9.40	2.17	.012[illegible]	.51	1.0
4	0.0628	0194	9.00	1.62	0002	.17	—0.2
5	1.0400	016[illegible]	7.10	6.13	.0012	.45	—1.5
6	0.2507	0290	7.00	1.64	0380	29	—0.2

SOLUTION AND DIFFUSION OF OXYGEN IN WATER.

There are few good data in regard to the rate of solution and diffusion of oxygen in water. Dr. Adney found that ten days were required to saturate with oxygen a sheet of water with a temperature of 15° C., 6 ft. in depth and deprived of all atmospheric gases.

The following experiments made by us give some idea of the rate of aeration of a quiet, shallow sheet of water. In the first, a small fish, weighing 25 grams, was immersed in an aquarium, 12 ins. in diameter, holding 7 ins. in depth of tap water. Experiments had shown that a fish of this size in this volume of water would reduce the dissolved oxygen primarily present about 0.26 part per 100,000 per day.

Table IV presents the amount of dissolved oxygen found in the water in aquarium on different days during a period of 34 days, together with a column showing the amount of oxygen consumed by the fish during that period. From this table it will be seen that oxygen was absorbed by the water at about the same rate that it was consumed by the fish.

TABLE IV.—DISSOLVED OXYGEN PRESENT AND CONSUMED.
(Parts per 100,000.)

	Dissolved oxygen	
Days—	In aquarium.	Used by fish.
1	.37	0.26
2	.40	0.52
3	.45	0.78
5	22	1.04
14	.49	3.18
22	.40	5.48
29	.50	7.08
34	.31	8.58

In a second experiment the dissolved oxygen in an aquarium of tap water was reduced to 0.11 part per 100,000 by the fish in this aquarium. The fish were then removed and the aquarium allowed to stand in the laboratory open to the air, and determinations of dissolved oxygen made from time to time. The results of these determinations are shown in Table V and it will be noticed that practically seven days were required for this water to again become saturated with oxygen.

TABLE V.—OXYGEN ABSORPTION FROM AIR.

Days—	Temperature (degrees F.)	Parts per 100,000.	Pct. of saturation.
2	68	0.38	41.3
3	65	0.72	74.7
4	64	0.76	79.2
5	66	0.80	85.0
6	67	0.88	94.5
7	70	0.94	104.0
10	65	1.00	105.0

During the last part of this experiment, "green growths" made their appearance in this water.

LIBERATION OF OXYGEN BY "GREEN GROWTHS."

It is well known that organisms containing chlorophyll give off free oxygen when growing in the light. Water in which organisms are plentiful is frequently found to be supersaturated with dissolved oxygen, as the organisms take the C from the CO_2 dissolved in the water and liberate part of the oxygen. When the dissolved CO_2 is all absorbed the bicarbonates are attacked and CO_2 abstracted, leaving carbonates. This happened several times in experiments here. On testing for CO_2 with phenolphthalein, a pink color showed the presence of carbonates, while in the tap water used in the various experiments the alkalinity was due to bicarbonates. In the experiments the "green growths" were largely Protococcus, Scenedesmus and smaller numbers of Monas. In one experiment an aquarium was filled with tap water and seeded with "green growth." The dissolved oxygen and CO_2 at the start were 0.18 and 1.0 part per 100,000, respectively. At the end of seven days they were 1.14 and 0.0 parts pere 100,000, respectively.

In another experiment a small fish was put into a tightly stoppered gallon bottle full of water containing these "green growths." If oxygen had not been liberated by the organisms during this experiment the oxygen primarily present would have been consumed by the fish in 38 hours. However, it lived eight days, and at its death at the end of this period dissolved oxygen was present in the water in greater volume than at the beginning of the experiment. The growth of organisms increased greatly during this period, and either the presence of these organisms or some body formed during their growth or decay was injurious to the fish.

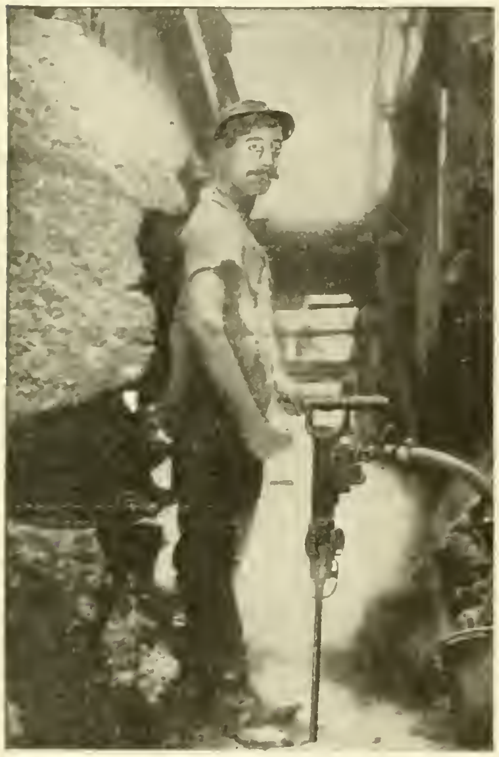


Fig. 1. View Showing Use of Steam-Driven Jackhamer Drill in Sewer Trench in Rock at New Britian, Conn.

In a similar experiment, made during cloudy weather when the oxygen-liberating power of the organisms was lowered, the fish present consumed the dissolved oxygen in four days.

In further experiments, not yet completed upon the effect of such "green growths" on sewage, bottles of sewage were seeded with organisms, then aerated for 24 hours and afterwards allowed to stand 24 hours without aeration. The average of 16 dissolved oxygen determinations then made showed 0.62 part per 100,000 of dissolved oxygen present in these sewages at the end of the period of aeration, and 1.01 parts per 100,000 after the sewage had stood unaerated during an additional period of 24 hours; in other words, the "green growth" supplied not only all the dissolved oxygen consumed by the sewage during the period of standing, but, furthermore, increased the volume present 0.39 part per 100,000.

It is probable that the chlorphyll-bearing organisms take a greater part in overcoming sewage pollution in surface waters than is at present generally realized.

Panama Canal Excavation in December.—The grand total of Canal excavation to January 1, 1914 was 215,158,272 cu. yds., leaving 16,891,728 cu. yds. remaining to be exvacated under the revised estimate of July 1, 1913. The total excavation for the month of December was 1,581,720 cu. yds.

Method and Cost of Drilling Blast Holes for Sewer in Rock, Using a Steam-Operated Jackhamer Drill.

The use of steam for operating hammer drills has been comparatively infrequent, compressed air usually being utilized for this purpose. In certain cases the use of steam may be more economical and more convenient, as where a steam plant is available and the magnitude of the work in hand does not warrant the additional expenditure of installing an air compressing plant. In such cases there is no good reason why steam should not be used if a drill adapted to its use is available. An application of the steam-operated hammer drill was recently made in sewer trenching in rock at New Britain, Conn. The method and cost of using a steam driven Jackhamer drill on this work was described by Charles P. Phelps in a recent issue of the Compressed Air Magazine from which the greater part of the information here given is taken.

The sewer under consideration is a 10-in. tile pipe sewer about 700 ft. long. The trench was about 15 ft. deep and 42 ins. wide. The sewer is located at the side of the road and parallels a storm sewer that is very close to the trench, but much nearer the surface. The proximity of the storm sewer made it necessary to exercise considerable care in cutting, bracing and blasting so as not to disturb the older sewer. The cut is principally in earth for the first 5 ft. and in trap rock for the lower 10 ft. The rock varies greatly in hardness, increasing the difficulty of drilling. Some of the rock encountered is extremely hard, and some is soft, oxidized trap.

Two Jackhamers were employed, their two operators accomplishing a much greater amount of work than the six drillers who had formerly carried on this work by hand methods. The drills were operated by steam at a pressure of nearly 80 lbs. per square inch. Practically the only difference between these drills and Jackhamers for air operation is in the piston, which is very slightly smaller in diameter in the steam drill to allow for expansion when the machine heats up. The drill handle being of wood, remains cool. A short pipe screwed into the drill outlet conducts the exhaust steam away from the operator, so that it is a very comfortable job to operate one of these drills even in warm weather. One particularly important feature of the Jackhamer is the automatic rotating device, by means of which the steel is rotated in the drill chuck, thus making it unnecessary to oscillate the drill by hand. The great advantage of this feature for trench work, where quarters are often cramped, will be at once apparent from an inspection of Fig. 1. When operated by air, a hollow steel is used with the Jackhamer. By means of a special form of valve, shown at A of Fig. 2, the air can be sent down into the hole through the hollow steel at the will of the operator, effectively cleaning it of cuttings. A steel retainer, B, also adds to the ease of manipulating the machine, as the steel and drill can be churned up and down in the hole while blowing out the cuttings, thus hastening the operation, the bit being held in place meanwhile by the special device B. With steam operation, however, the hole-cleaning arrangement did not prove to be very satisfactory in this rock, so the hole was kept full of water poured in from a pail. The cuttings were scooped by means of a tree branch having a smashed end. The branch was driven into the hole and withdrawn quickly, this pump-like action forcing the water and cuttings out of the hole in a satisfactory manner.

The sides of the 5 ft. earth cut were shored up with planking. The first cut made in the rock was about 4 ft. deep to get below the level of the adjoining sewer. The remaining 6 ft. was bored in two cuts of 3 ft. each. The spacing of the holes averaged about 2½ ft. both lengthwise and crosswise.

Three changes of steel were used per cut. The starting bit was 2 ins. in diameter, the second 1¾ ins. and the bottoming bit 1½ ins. After drilling a hole to bottom, a wooden plug was inserted in it to keep out dirt, etc.

A six-point bit was employed for drilling the softer rock, but in the harder rock a cross bit was used.

Four sticks of 60 per cent dynamite were used per hole. At the beginning of the work 40 per cent dynamite was used, but it proved not to have sufficient strength for this kind of rock.

The number of laborers on this work varied at different stages of the work. The foreman had general charge and looked after the boiler as well. There were two drill runners. The force of muckers who removed the spoil with pick and shovel and replaced it consisted at various times of from 15 to 40 men.

There was one nine-hour shift per day, excepting on Saturdays, when the shift was of five hours' duration. The rates of wages were as follows:

Drill runners	$2.50 per day
Laborers	2.00 per day
Blacksmith	2.25 per day

The speed of drilling varied greatly owing to the varying hardness of the rock. Sometimes the performance of the two drills was as high as 80 ft. per 9-hour shift, and sometimes it was as low as 50 ft., but the average drilling performance was 60 ft. of hole per day, or more, which is equivalent to about 7 ft. of hole per hour in trap rock of average hardness. The very poorest record for a single day's work with one drill was 10 ft. of hole. Taking the average figure of 30 ft. per day per drill, the labor cost of drilling would be 8⅓ cts. per foot of hole.

The blacksmith and his helper devoted about seven hours of the nine sharpening the steels. For both drills not more than 8 ins. of steel broke off and caused trouble during three months of work.

The power plant for this job consisted of a portable Ames steam boiler carrying about 80 lbs. pressure. The water was fed to it by an injector, the consumption averaging about 192 gals. per day. During the first eleven weeks of constant operation about 1½ tons of soft coal were consumed.

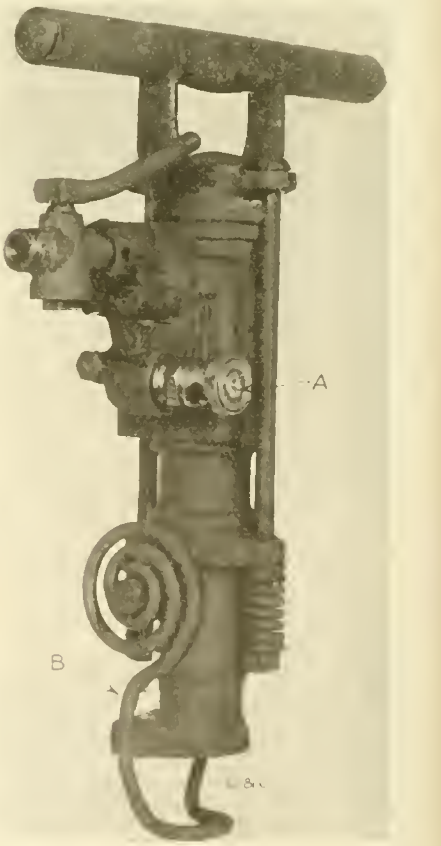


Fig. 2. View of Power End of Jackhamer Drill.

Leading from the boiler was a 1-in. iron pipe which crossed the road and connected with several lengths of pipe of the same diameter, laid along the ground beside the trench. This pipe had tees at convenient distances for connecting the drill feeds. When it became necessary, with the advance of the work, to move the boiler along, the back lengths of pipe were unscrewed and connected to the front. The drill feeds were $\frac{3}{4}$ in. rubber steam hose.

Railways

Methods of Constructing a Short Rock Tunnel on the New Low Grade Line Between Milwaukee and Sparta, Wis., Chicago & Northwestern Ry.

Contributed by L. J. Putnam, Principal Assistant Engineer, Chicago, Ill.

GENERAL DESCRIPTION.

During the construction of the Chicago & Northwestern Ry. Co.'s new low-grade line across Wisconsin from Milwaukee to Sparta, a short tunnel was driven through the range of hills, which forms the divide between the waters of the Wisconsin River and those of the Mississippi River throughout this district. This tunnel, while short and presenting no very unusual features is nevertheless, thought to be of more than passing interest as an example of practical tunneling in a small-sized tunnel, which would not warrant the installation of elaborate and expensive machinery and methods.

The tunnel is located just west of the village of Tunnel City, Wis., which owes its name and origin to the little settlement which was started at that point during the construction of the Chicago, Milwaukee & St. Paul Ry. Co.'s first tunnel through the same range of hills early in the sixties. This first tunnel of the C. M. & St. P. Ry. was of small bore and only timber lined and was abandoned in 1873, when their second single track tunnel, which is still in use, was driven alongside. The C. & N. W. Ry. tunnel is north of, but not parallel to, the C., M. & St. P. Ry. tunnel and distant about 200 ft. at the west portal and 400 ft. at the east portal. The C. M. & St. P. Ry. tunnel is 1,330 ft. long, single track, brick lined, while the new C. & N. W. Ry. tunnel is 1,295 ft. long, double track section and lined with concrete.

It was first intended to make the tunnel about 2,700 ft. long, or from Station 814 to Station 841, at which points the cutting would have been approximately 50 ft., as shown on the accompanying profile, Fig. 1. After soundings to rock were completed, and further study given the matter, it was decided to shorten it to the length of 1,295 ft., although by so doing, the actual cost per lineal foot was somewhat increased in the open excavation as compared with tunnel. The improvement in light and ventilation on the shorter tunnel was considered worth the small additional first cost. As thus shortened, the east portal is located just east of the gully near Station 825, so that the cross drainage which amounts to considerable at times, is taken care of in its natural course without any diversion.

The material encountered throughout the length of the tunnel and practically the entire east portal cut was the soft Potsdam or basal sandstone. The west portal cut was largely loose sand, the debris from the original sandstone ledge. This Potsdam sandstone is a very soft rock, standing fairly well in nearly vertical faces, but where these are exposed in nature, they become wind worn into fantastic shapes and have formed many interesting and beautiful buttes, throughout this particular region. When blasted, this rock breaks up into its orginial sand grains, only occasional strata being hard enough to retain any resemblance to rock after blasting. The blasted rock is therefore only a pure white sand handled with the greatest ease.

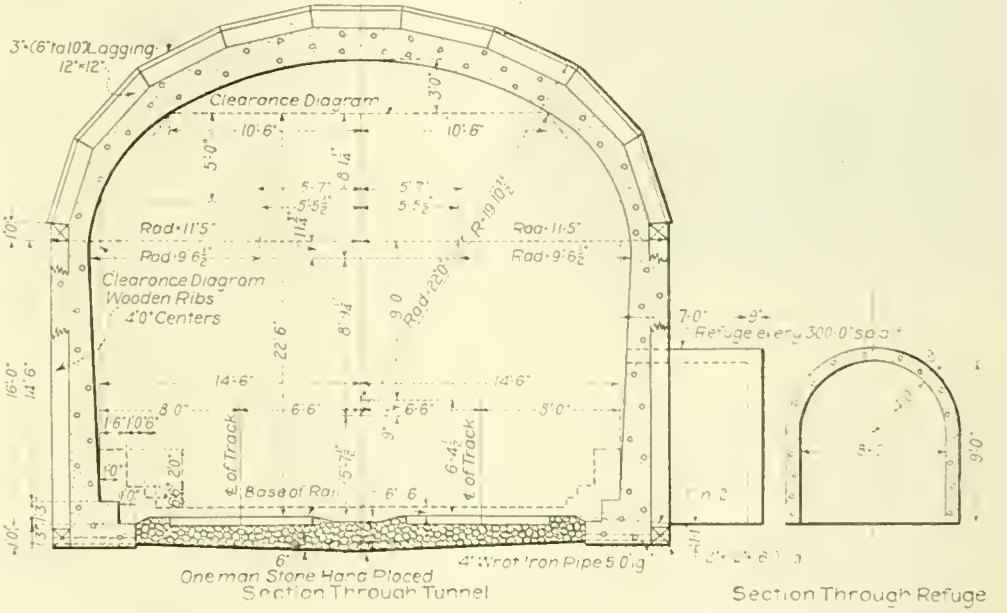


Fig. 2. Section of Chicago & Northwestern Ry. Tunnel.

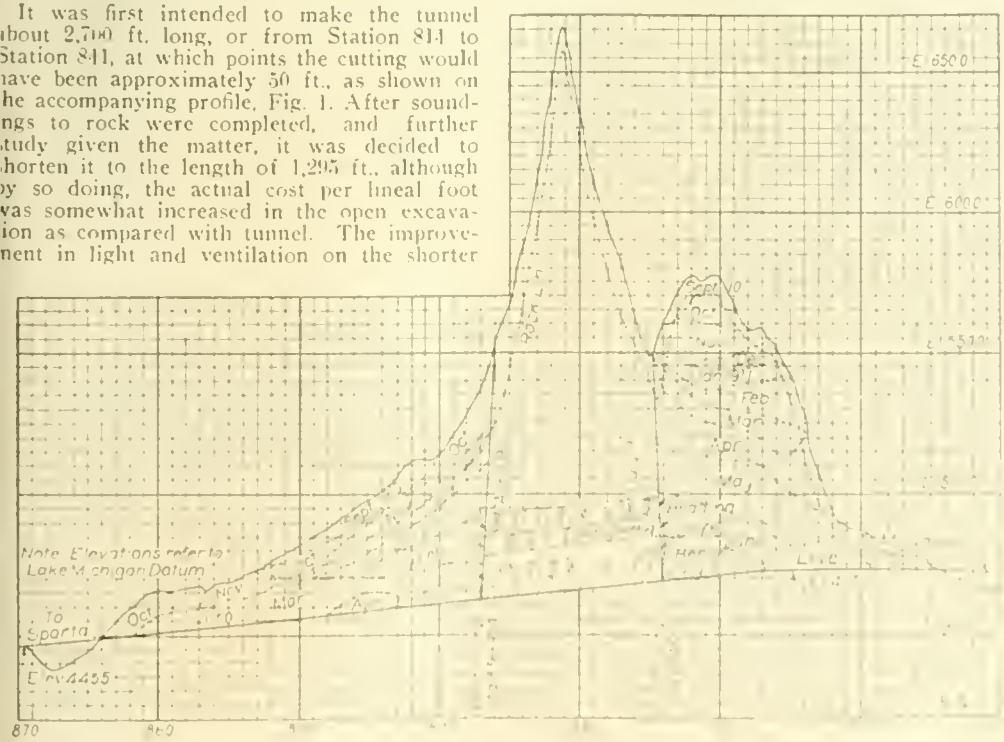


Fig. 1. Progress Profile of Tunnel for Chicago & Northwestern Ry. Low Grade Line Between Milwaukee and Sparta, Wis.

ENGINEERING WORK.

The tunnel is entirely tangent and has a continuous 0.5 per cent grade through its entire length. The engineering problems as to line and grade were therefore of the simplest possible. Three permanent monuments were established on the center line before work was started. These were made of 2-in gas pipe plugged with brass and having a removable cap. The pipes were set in concrete monuments carried down below frost and about 16 ins. square. These concrete shafts were cast in place, simply filling holes dug in the earth. One monument was on the apex of the hill and the others about 3,000 ft. distant in either direction, being on the subtangents to the curves east and west.

After the portal cuts were completed, temporary monuments were established back from each portal far enough to be convenient and providing an unobstructed line of sight to the monument on the hill-top. This line was easily and accurately projected into the tunnel from either end. Each 16-ft length of wall place (longitudinal timber at spring line) was accurately placed for line and grade and no other instrument work was required during the excavation. The wall plate was fixed for alignment by off-setting a line on the bench and projecting it into the drifts. Levels were carried along readily in the drifts, care being taken to check back frequently to bench marks at the portals which were previously accurately established from series of levels run over the hill and also from levels run through the C. M. & St. P. tunnel. The closing error in levels was only 0.02 ft. and alignment was practically perfect. An ordinary set of railroad instruments was used and points in the tunnel were clearly seen by the aid of candles or hand torches.

TUNNEL SECTION.

The section of this double track tunnel is shown by Fig. 2. It is 29 ft. wide in the clear at a point 3 ft. above the top of rail with a slight batter on the side walls. The vertical height is 25 ft. from top of rail to top of arch, providing 22 ft. clearance on both tracks at a distance of 4 ft. from the center line of the tracks. The roof is a three-centered arch. Lining is of concrete which is 2 ft. or over in thickness between the 12 x 12-in. timber sets and 1 ft. or more over all timbers. The section in Fig. 2 indicates the concrete lining thicker over the crown of the arch. This was so designed in order that there would be one foot of concrete over the timbers after the settlement of timbering had taken place. Later development proved this precaution to be well taken as in some extreme cases the timbering settled to such an extent as to reduce the thickness of concrete lining slightly and in a few special cases it was necessary to remove part of the timber segments as referred to later.

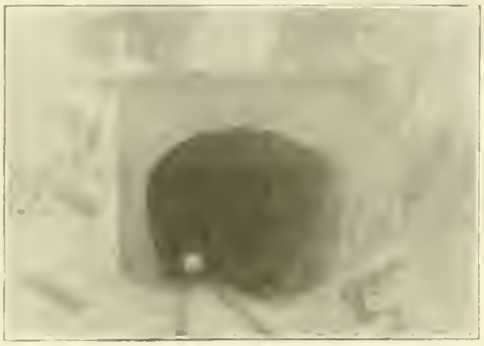


Fig. 3. Portal of Chicago & Northwestern Ry. Tunnel.

Refuges 8 ft. wide and 7 ft. deep—large enough for hand cars—were constructed at intervals of 300 ft. alternating between sides.

The portals of the tunnel are of concrete with simple design as shown in Fig. 3, a photograph of the east portal after completion. Drain holes were provided through the side walls just above the base of rail at 16 ft. intervals. It is doubtful, however, whether these will serve any useful purpose as the tunnel is dry throughout except for about 100 ft. in length near the east portal.

APPROACH CUTS.

The excavation of the approach cuts was started in September, 1910, at both ends, one steam shovel working in each cut. Narrow gauge track and dump cars were used in each case and trains were handled by dinkey engines. The material from the west cut was hauled a maximum distance of five miles but the haul from the east cut was never more than one mile. The yardage handled from the east portal cut was 320,000 cu. yds. exclusive of 12,000 cu. yds. from the tunnel bore. The larger part of this being waste, the upper levels were first wasted in a high level gulch, to the north of the cut. The maximum cut at the upper slope was 140 ft. and the handling of the material down from this height would have been very difficult except for the high level waste ground. At the west cut the grade of the ground surface was not excessive and material was handled to the fill from the start without unusual difficulty.

Previous to grading work soundings were taken near each slope line at every station or oftener so as to locate the rock elevation. This work was done by means of a common well drill outfit. Holes 4 ins. in diameter were put down into firm rock. Casing was required, through sand but it was readily recovered and holes allowed to fill after measurements were obtained. From these test holes, sections were accurately plotted and slope stakes set. Slopes 1½ to 1 were used in earth and ½ to 1 in solid ledge, but leaving a horizontal berm of 10 ft. width on the surface of the ledge. These plotted sections indicated to contractors, just where to expect the ledge and shovel cuts were arranged accordingly. After the rock was stripped in any cut of the shovel, drills were started and in the following cut, shooting was done immediately in advance of the shovel. Drill holes were ordinarily 2½ ins. in diameter and spaced 10 ft. apart in each direction but staggered. Steam drills mounted on tripods, taking steam from portable horizontal boilers were used and the progress of drilling was rapid. Two to four drills were required to keep ahead of a steam shovel.

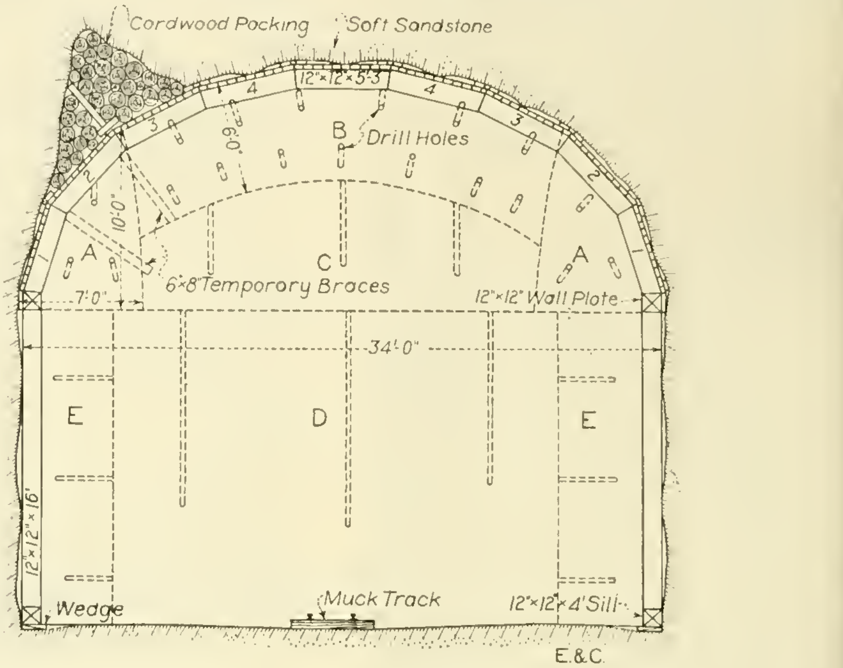


Fig. 4. Section Showing Sequence of Excavation Chicago & Northwestern Ry. Tunnel.

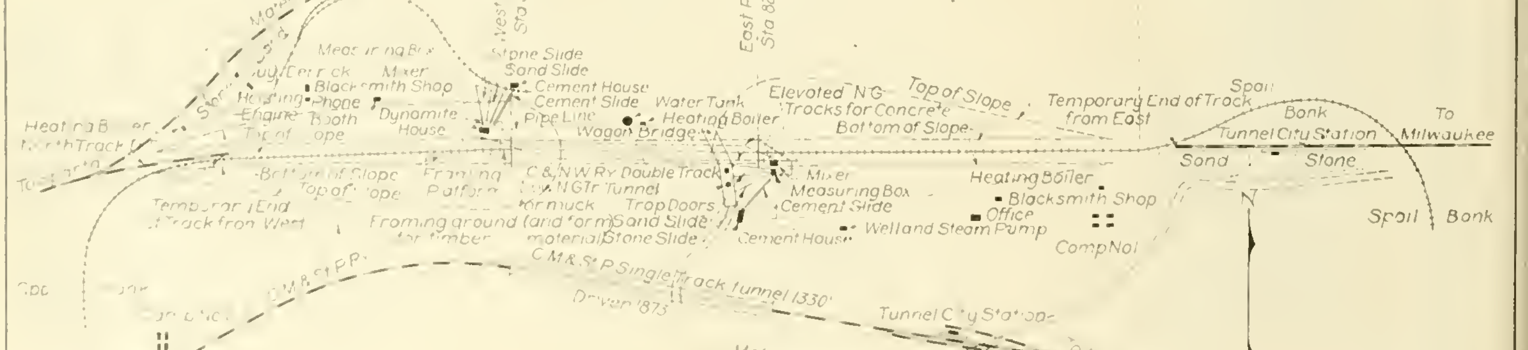


Fig. 5. Plant Layout Plan, Chicago & Northwestern Ry. Tunnel.

TUNNEL EXCAVATION AND TIMBERING.

The method and sequence of tunnel driving is indicated in Figs. 4 and 5 and the progress of the work is shown on the profile, Fig. 1; progress being indicated separately for the side drifts, the top heading and the bench. On account of the yardage in the west open cut being less than in the east, the portal face down to the level of the side drifts was exposed at that end first permitting the west drifts to start there in March of 1911, while at the east portal, the drifts could not be started until July of the same year. This resulted in about two-thirds of the tunnel being driven from the west end. In Fig. 4, the location and comparative size of drifts, heading and bench are shown. The drifts designated *A*, were about 7 ft. wide at the bottom which was on a level with the under side of the wall plates, but narrowing at the top in the form of a Gothic arch. Ordinarily three drill holes were used in positions as indicated and inclined slightly outward from the center. These were 2-in. holes about 4 ft. in depth. Light charges of dynamite were used so as not to shatter the soft rock outside of the tunnel section. Muck from the drifts was wheeled back in wheel-barrows and dumped over the end of the bench where it was loaded on dump cars along with other muck

Drifts were usually about 100 ft. in advance of the heading, giving a wheelbarrow haul of about 150 ft. The wall plates were laid in the drifts in 16-ft. sections as the drifts progressed and were accurately lined and surfaced, resting on the rock or on blocks. Special pains were taken to have solid rock outside of the wall plates to take the horizontal thrust of the arch. If the rock was accidentally shattered, back of the plates, substantial blocking was placed opposite each future timber rib. Joints in wall plates were made with a half-splice 16 ins. long bolted together and midway between ribs. In some cases the roof of the drifts caved badly even though special precautions were constantly taken and in one extreme case progressive sloughing off from the roof continued for 20 ft. above the proper top of the drift before it could be arrested. This opening was subsequently packed with cordwood as shown in Fig. 4. Where there was difficulty of this kind the two lower segments of the rib with the accompanying lagging were placed in the drift, the joints being supported in proper position with temporary stulls until the balance of the ribs were in place. Ordinarily, this was not necessary and the entire rib was raised after the full heading, *B*, was opened up.

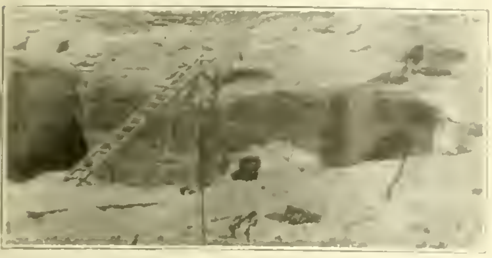


Fig. 6. View Showing Side Drifts at East Portal, Chicago & Northwestern Ry. Tunnel.

Section *B* was removed as the second step and was usually about 100 ft. in the rear of the drifts. Drill holes were located about as shown and were approximately 4 ft. deep, so that after each blast the roof could be trimmed and a 4-ft. section of timbering placed, the ribs being 4 ft. center to center as shown. Section *B* was kept about 6 ft. high, so that men could stand conveniently to drill the face and to trim the roof and erect he timber. The rib segments were carefully framed to patterns and when placed, the proper position of each joint was readily obtained by bringing it to a fit. Ends of segments were doweled together excepting the op segment, which was placed last and toe nailed. After the rib was set up and lagging n place, blocks or wedges were used at the oints to bring them to a firm bearing against he rock. Openings of any considerable size above the lagging were packed with plank or cordwood, but in most cases the roof could be trimmed so accurately that it made a sufficiently close fit. The roof usually stood securely for the length of time necessary to get the rib and lagging in place. If indications were that it would not, only one hole was shot at a time and temporary struts between ribs were placed until the rib could be placed. For a short distance in the worst place, no drilling or blasting in the heading was done, all rock being picked out by hand and the roof immediately shored. Muck rom the heading was wheeled back as in he case of the drifts and dumped over to the hovelers below.

Section *C* was removed about 30 ft. in the ear of the heading, the muck removed was argely shot down over the end of the bench without shoveling. No difficulty was encountered on this section.

After Section *C* was removed there was ufficient head room so that long-steel drills ould be used with rather heavy charges of xplosive. The center hole in *D* was three to our feet in advance of the other holes so as to keep the end of the bench in crescent hape to facilitate removal of Section *E*.

Sections *E* were removed in 4 ft. advances. Men stood on the rock slope of the bench and picked the material down or used light charges of explosive in drill holes as necessary. The wall was thus trimmed out in 4-ft advances so as to clear the space for the next plumb post and still leaving solid rock to support the wall plate just in advance of the post. In this way the wall plate was never unsupported under more than one rib at a time. With Section *E* removed for 4 ft., the new plumb post was set on its 4-ft. sill and wedged up tightly. Plumb posts were always directly underneath the foot of the arch ribs. No lagging was required below the wall plates in any cases.

A narrow gage track was laid on or near the center line of the tunnel. A short siding was provided to hold one train of cars near the working face. Empty cars were pushed out from this siding to the face, loaded and dropped back to clear the next car. Six to eight of these 1-yd. cars were thus assembled into a train which was hauled out by team. Twelve to fourteen men loaded the cars and shunted them about as indicated. Shoveling being so very easy, only a few minutes were required to load a car and from 150 to 200 cars per 24-hour day could be readily handled at one heading. The location of this narrow-gage track, spoil banks, etc., are shown in Fig. 5.

In starting at the portals, Section *B* could not be driven from the open face on account of the great danger of breaking down of the overhanging rock. To avoid this danger a cross drift about 6 ft. wide was carried across between the side drifts and about 32 ft. in from the portal. In this drift two ribs with the lagging were placed. Work on the heading then progressed outward to the portal. This afforded protection at all times to the men and by placing the ribs at 2 ft. centers near the portal, very little of the roof was left unsupported and no trouble ensued. This of course made the start slow as all muck had to be removed through the side drifts. This slow period, however, was passed while the last cuts of the steam shovel were being made in the open cuts so that when the shovel was through, everything was ready for full tunnel driving. A view of the side drifts at the east portal is shown in Fig. 6.

The ventilation became rather poor, due partly to the fact that this was on an ascending grade. To improve conditions the west drifts were crowded ahead and brought to a meeting with the east portal drifts even though the muck had to be wheeled for a considerable distance. When the drifts met a very strong natural air current was set up providing ample ventilation. In fact, curtains had to be placed so as to check the current partially in order to keep lights burning.

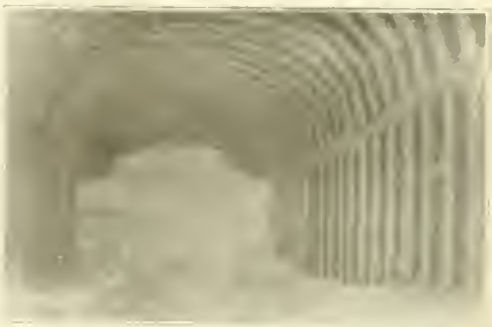


Fig. 7. Timbering of the Chicago & Northwestern Ry. Tunnel.

Candles were used principally for the illumination at the working face and banjo torches were used to light the teamway. The large section of the tunnel and the white color of the rock, assisted greatly in the distribution of the light so that for 300 ft., or more at each end, very little artificial light was required by the day shift.

A variety of drills was used. In the softer strata twist drills were used to very good advantage. These were simply carpenters' augers with the lips remodeled. In many places these could be bored into the rock as fast as a carpenter would bore into wood. In harder rock the common rock drills driven with sledges, or churn drills in the case of the bench holes, were used. No water was required.

The progress after the work was organized averaged 4 ft. per 24 hours at each heading.

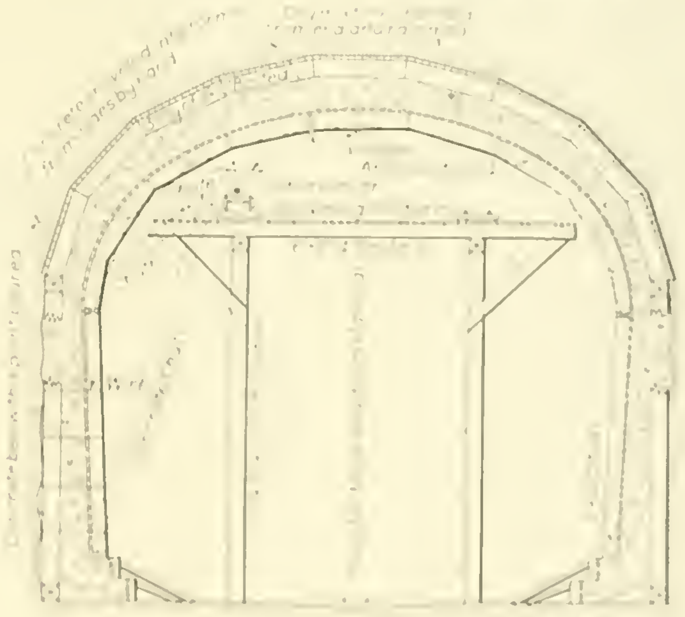


Fig. 8. Section Showing Method of Placing Concrete Lining, Chicago & Northwestern Ry. Tunnel.

[illegible] [illegible] dynamite was used throughout [illegible] [illegible] tunnel. No artificial ventilation was necessary and after a blast, a ten minute [illegible] sufficed to clear the working face sufficiently for work to be resumed. Firing was done by fuse throughout. When the west heading was in about 600 ft. [illegible] [illegible] 16 [illegible] [illegible] [illegible] the low portions [illegible] [illegible] at the west heading.

The settlement of the timber ribs [illegible] of the working face showed that the roof broke [illegible] behind irregularly and often in very heavy masses which threw concentrated loads on the ribs at various points. If this had been fully anticipated before work was started, it is quite probable that the design of the arch section would have been somewhat changed. When this became apparent, it was too late to make any changes. The joint at the upper end of the third segment gave the most trouble, partly because for quite a distance there was a fault in the rock at this location. The settlement of this joint became so pronounced in some cases that long shores of cedar piles were placed to support it. These shores were generally left in place until after the concrete lining was in and set up. In such cases boxes were placed around the tops of the piles. These boxes were shaped like a truncated pyramid, big end up, and after the piling was removed the boxing was taken out and the hole concreted.

In five or six instances, the timber segments settled so as to leave only a few inches between them and the line of the finished concrete lining. In these cases T-rails were bent to fit the location and one placed on either side of the rib. After the concrete in the arch had been brought up well onto the rail ends and set up, the timbers were removed and the lagging supported by the rails giving practically the same section of concrete.

At one point near the east portal, three ribs with the lagging fell to the floor of the tunnel, bringing with them 100 cu. yds. or more of rock from the roof. The portion which fell was back of the working face and gave sufficient warning so no one was caught. In this particular case sufficient care had not been taken to see that all joints were properly wedged up, resulting in a deformation of the rib under heavy concentrated load. The occurrance proved a valuable lesson not thereafter forgotten. The opening was very difficult to fill and the replacing of the ribs and lagging and packing of the space occupied about four days' time. This accident was the only one during the progress of the work and the tunnel was completed without the loss of any life or the serious injury to any workman. Figure 7 is a view showing the timber lining and something of the method of excavation.

CONCRETE LINING

On account of interference between the driving and the lining forces, the concrete lining was not started until the driving of the tunnel was about two-thirds complete. One mixer was placed at each portal and about level with the top of the tunnel so that mixed material was dumped by gravity directly into small cars on the runways. Figure 8 shows this runway platform just above the spring line of the arch. All material being chuted down to elevations below while for the portion above the mixed concrete was dumped on the platform and shoveled into the forms.

Figure 5 also shows the general arrangement of the contractor's plant and the method of operation. It will be noted that at the west portal stone and sand were unloaded by a clam-shell operated from a guy derrick. The same clamshell reloaded the material from the stock pile into dump cars which were then hauled by team up to a chute at the portal above the mixer. Very little hand shoveling was thus required at the west end. Since more than three-fourths of the concrete was placed by the west plant, it was thought that the small amount handled at the east portal would not justify the expense of installing machinery. Consequently at the east workings, concrete material was unloaded from cars and reloaded into wagons by hand. The wagons were hauled by team on a long detour to above the portal and dumped through a bridge into the chute above the mixer as shown.

At both ends the standard gauge track had been laid up as closely as possible to the portals before the concreting was started so all material was switched to the unloading points by the railway company on its own track.

A very good quality of bank run gravel was supplied by the railway company from its gravel pit, from which ballast was being obtained. This material was loaded at the pit by the steam shovel at small expense. The bank run material screened out about 50 per cent sand and sufficient stone was added to make the final mix a 1:2:4 concrete.

The portals were completed first and the lining carried inward from the ends joining about one-fourth of the distance from the east portal as stated. Footing concrete was placed well ahead so as not to interfere with the form work. Side forms were of the usual construction of vertical studs and horizontal matched sheathing. The studs were capped with a plate on which the arch centers rested. The side wall studs were placed opposite the plumb posts and wired through to them in the usual manner. Bulkheads were built at 16 ft. intervals and one 16-ft. length concreted every working day. After a section of side wall was poured and set, centers were erected and a 16-ft. length of arch was also completed daily, but of course the side walls were always about two days in advance of the arch.

The method of concreting the arch above the level of the working platforms was to dump the concrete mixture on the platform and shovel from there into the forms. Strips of lagging were placed as fast as the filling of the forms progressed. Care was of course taken to see that no voids were left around the timbers but with the medium wet mixture used very little spading or tamping was required. When the lagging had been carried up both sides so that only about 4 ft. in width of space remained, the balance of the concrete was placed from the end and rammed. For this key section, lagging was cut to 4-ft. lengths reaching only from one center to the next. Concrete, thus had to be handled in end only 4 ft. permitting proper packing and tamping.

The usual procedure was to fill 16 ft. of the side wall forms and the portion of the arch section below the platform during the day shift and to complete the arch section during the night shift. This arrangement permitted running the heavy yardage during the day and doing the slow work and light yardage at night. Only a small night shift was thus required and the hauling of material by team at night was wholly avoided.

The side wall forms were stripped and the same material re-used ahead as far as possible. Since there was no load on the arch, the centers could also be struck after 48 hours and these also were re-used over and over. To facilitate the striking and moving of the centers wedges were placed between the ends of each ply of plank composing the center at a joint near the crown, and the centers were only bolted together at the laps adjacent. Bolts first being removed, the wedges were knocked out, permitting ready removal of the center in two sections.

The concrete lining was placed largely in November and December and an unusually cold period during these months made it necessary to take precautions to insure the best quality of concrete. Sand and stone were heated by steam boxes at the unloading points and also in the slides when necessary. Water was heated in the supply tank on the top of the hill and both ends of the tunnel were closed with canvas curtains. With the curtains down and the large number of torches and lights in use inside, it was unnecessary to provide any other interior heat.

The concrete lining was completed and forms and machinery out of the way of track laying on Jan. 3, 1912. Track was laid through the tunnel and rock ballast placed so that the line was opened for traffic on Jan. 10, 1912.

Mr. E. C. Carter was Chief Engineer, Mr. W. H. Finley was Assistant Chief Engineer, and the writer was Resident Engineer for the railway company. Winston Bros. Co. of Minneapolis were the general contractors, and Bates & Rogers Construction Co., Chicago, were sub-contractors for the concrete lining.

The Meaning of Government Regulation of Railroads.

Sirs: It is suggested for discussion: Does not government regulation mean as a matter of fact government ownership in whole or in part—that the logical end of government regulation is government ownership, absolute and complete?

Left to themselves, the railroads earn what can.

Now comes the court and limits them to fair return on the value of the property in service. "Fair" is an indeterminate word, but nevertheless it is proposed to set a limit or limits. To limit returns, rates are limited. If a property would earn 8 per cent under competitive conditions, and the government cuts it down to 6 per cent, then the value of the property which would have accrued from the 2 per cent is taken away from the owners and distributed among the public. The owners have the remaining value. The ownership of the value as it originally existed has been divided and distributed part to the public and part to the original owners.

The essential features of ownership may be considered as profits, losses and responsibility. Any actual limitation of profits amounts to the destruction or taking away of value. In this case it is not destroyed, but taken away and given to the public. Evidently the process of lowering rates cannot continue without a wholesale destruction and driving away of capital, unless losses are correspondingly limited. Of course the capital invested will not be really destroyed. There is simply a transfer of an essential feature of ownership from the stockholders to the people at large.

If losses are correspondingly limited and a minimum return assured, we then have two limits as to profits—a high limit and a low limit. The value to the owners in such case may remain unchanged. To preserve this value, however, the low limit must be raised, just as the high limit is cut down; until finally with the increase of regulation, both as to rates and operation, we shall arrive at a point where the government and public own the railroads, and the stockholders own a government bond.

Any regulation of rates short of this limit will constitute partial ownership by the public; an effective transfer of values where profits are limited, or an approach to complete government ownership where profits are both limited and guaranteed.

Very truly yours,

C. P. Howard, M. Am. Soc. C. E.

Chicago, Ill., January 17, 1914.

(Some of the essentials of "ownership" used in its popular sense (with the meaning of the man who asked, "Shall I not do as I please with mine own?") have been for some time passing from railway stockholders to the commonwealth through the powers vested by the public in the Interstate Commerce Commission.

If regulation is pushed to the point of actually discouraging present owners by large risks and small returns government operation may at any time become necessary and thus make public ownership virtually complete.

It is conceivable that government representatives or officials may some day be capable of handling economically the enormous business of the railways; but it is to be hoped that this revolution (supposing it to be inevitable) will be gradually and quietly brought about and that, meantime, the interested cooperation of those who now own and manage our transportation systems may be held by fair treatment and payment proportionate to performance.—Editors.)

The main line of the Grand Trunk Pacific Railway has now been completed to a point 303 miles east of Prince Rupert, B. C., and trains are in operation as far as Mile 301, Rose Lake. Steel has been laid to Mile 1240 west of Winnipeg or 145 miles west of Tete Jaune.

Engineering and Contracting

Devoted to the Economics of Civil Engineering Design
and to Methods and Cost of Construction

Volume XLI. CHICAGO, ILL., FEBRUARY 11, 1914. Number 6.

Volume and Cost of Water Required for Concrete Road Construction.

The contractor for building and general concrete work seldom has need to concern himself much about the source and cost of his water supply. Mixing water only is required, say 50 gals. per cubic yard. There is a water cost of course but it is substantially cost at tap for most city work, and, elsewhere, cost of pumping or carrying only a short distance. Conditions are greatly different when a concrete road is the task in hand. Here, securing and delivering water become problems in the most favorable locations and they become real difficulties when the work is in dry country. In arid regions, water for concrete road construction is veritably a precious thing. A hint of these facts will be found in this issue in two of the articles on concrete road construction but the facts deserve further emphasis.

Concrete contractors who undertake concrete road work for the first time will discover that the volume of water required is unsuspectedly great. No more mixing water is required for concrete for road than for concrete for other work but in road construction there are two other demands for water and each is greater than is the demand for mixing water. Water is employed in the construction of the earth sub-base for the concrete slab and water is employed to cure the concrete slab after it is laid. The supply and delivery of water for sprinkling sub-grade and curing pavement are as much a demand as are the supply and delivery of water for mixing the concrete; the only difference is that the volumes of water required for sprinkling and curing are not so nearly constant as is the volume required for mixing.

Probably a fair average of the volume of water required for mixing concrete for road work is 50 gals. per cubic yard. This volume will seldom be decreased but at times it may be increased though for road work the increase is not likely to be large. Practically, the volume of water required for mixing in any case remains constant regardless of natural conditions. On the contrary the nature of the soil, the season, the climate and the weather all influence the volume of water that will be required for sprinkling sub-grade and for curing the pavement. Specific figures are scarce. In the article in this issue on California State Road work the author gives no quantities but assumes in distributing his water costs that equal volumes were employed for mixing, for wetting sub-grade, and for curing pavement. Assuming 50 gals. for mixing, the total volume of water consumed for all three purposes was 150 gals. per cubic yard of concrete pavement laid. This figure is, we imagine, not far from a mean value. On concrete road work in California known to the editors maximum figures gave per cubic yard of concrete 100 gals. for wetting sub-grade, 50 gals. for mixing concrete and 80 gals. for curing pavement, a total of 230 gals. or say one ton of water per cubic yard of concrete pavement. Minimum figures reduce the total volume some 75 per cent.

Not including water for steam, camp use, etc., concrete road construction requires from 100 gals. to 250 gals. of water per cubic yard of concrete in place. These quantities alone are quite sufficient to demonstrate the importance of the cost item for water supply. On the California State Road work already mentioned water supply and delivery cost about $18 per day per 100 cu. yds. of concrete pavement laid or virtually 18 cts. per cubic yard. In this work the water was pumped and piped along the work, and in all cases but one, pumping was possible from natural streams. The one exception, however is worth noting because in much semi-arid and arid country the construction of wells for water supply will be the rule and not the exception.

The necessity of installing virtually a water supply system for concrete road work is likely to come as a surprise to the contractor whose concrete construction has been in cities and where water is directly available and also where about the only water required is that used in mixing. It is a necessity, however, and one that is not confined to dry country. In the article in this issue on the La Salle, Ill., concrete road, the author states that it is the custom in such work of the Illinois Highway Commission to "construct a complete system of pipe lines to which water may be furnished by city pressure or by a pressure pump operated by a gasoline engine." While a pumping and pipe line system may seem a rather expensive item one has only to figure for a bit, tank wagon hauling costs to see that the pumping system may often be an investment well warranted. A hauling cost of 25 cts. per ton mile is not a high cost on many works of construction and 240 gals. of water weigh a ton. The editors have knowledge of pieces of concrete road work in dry country where the cost of hauling water amounted to 25 cts. per mile hauled per cubic yard of concrete in pavement.

In closing the need may well be noted for the publication of more precise figures on the volume of water required for concrete road construction. Also there may be noted the importance of studies to determine the method of curing concrete pavement that is most economical in water consumption. The articles in this issue and already referred to offer items of interest in this determination.

Stresses in Concrete Roads and Required Thickness of Slabs.

The required thickness of concrete roads and concrete foundations for various types of pavements has not been given a great amount of real study. In most cases engineers have decided upon a certain thickness after noting what others have been using—the "others" probably had no better data to guide them. By reviewing specifications for work done in various localities one must conclude that the soil and traffic conditions have had little to do with the determination of the thickness of the slab. From the nature of the problem it is impossible to get a definite solution for any particular case without making some assumptions, but careful investigations and tests should tend to reduce the number of assumptions required. In this issue there is an article on the approximate stresses in and required thickness of a continuous slab supported on a yielding foundation and subjected to a concentrated load. Assumptions are made in the investigation as to the distribution of the load over the foundation material. The paper is offered as a means of stimulating discussion and investigation rather than as furnishing working formulas. Some of us are too much inclined either to accept the conclusions of investigations without following out the method used, or to dismiss any mathematical investigation without giving it due consideration. The great call for empirical formulas which can be applied without much effort on the part of the engineer is an evidence of this tendency. A rigid mathematical investigation which is based upon the same assumptions as one which only considers the results of observation is certainly of greater value. Of course both are useless if the conclusions are entirely erroneous. It is essential that we develop better methods of determining the required thickness of concrete roads. We will all agree that the use of a slab which is too thin to give efficient service is poor practice, but is not the error just as serious if we make the slab thicker than is required to meet soil and traffic conditions? In the first case the pavement will have a short life, but in the latter case many of us will not have a pavement at all. In constructing roads and pavements the question of safety is not one which demands great attention; that of cost is paramount.

The Latest Concrete Dam Failure and Its Lesson.

In an editorial published in this journal of Dec. 11, 1912, we commented upon the failure of the concrete dam at Austin, Pa., and stated that the failure of this structure, in connection with all the attendant circumstances, constituted a forceful argument in support of the desirability of the state control of the design, construction and operation of dams. The latest concrete dam failure, that of the Stony River dam, near Dobbin, W. Va., which occurred on January 15 of this year, is an additional indication of the need of state censorship of dam construction. The two failures mentioned present such strong contrasts, in many important particulars, that considered together they go far toward making a complete exposition of the reasons why state control of the engineering and operation of dams is so desirable. With this fact in mind, we are here reviewing very briefly what we previously pointed out relative to the Austin dam failure before taking up the discussion of the recent failure of the Stony River dam.

The designer of the Austin dam had two limitations placed upon him by the owner, namely minimum cost and maximum height. As revealed by the failure of the dam the designer's mistake in building this dam was in trusting to the rock foundation as being impervious. The cause of the failure was the sliding of one stratum of the underlying sandstone upon another, after the intermediate layer of shale and gravel had been softened by the water which seeped through it. The most striking fact in connection with the Austin dam disaster was that the structure failed, in an engineering sense, on Jan. 23, 1910, and was demolished on Sept. 30, 1911, over 18 months later. After the failure, when the dam dropped down some 6 ins. at the toe near the center of the spillway, and slid out at the spillway about 18 ins., the structure was not really a dam, but was a vertical concrete wall broken in five or six distinct and separate sections. Water was impounded by this broken concrete wall between the dates mentioned. During this period water continued to leak through the underlying strata and this leakage going on at great pressure, that due to a head ranging from 25 to 32 ft., washed much of the shale and gravel away, thereby leaving the heavy concrete walls with a foundation which gradually weakened until the final collapse came.

The failure of the Stony River dam was only partial in that but 5 of 66 15-ft. bays of an Ambursen type of reinforced concrete dam collapsed; the dam was 1,065 ft. in total length, including the earth embankments at the ends of the concrete section, and 51 ft. high at its deepest section. This failure caused very little property damage, aside from the damage to the dam itself, and no lives were lost. The dam as built was chosen in preference to three alternative designs for hollow reinforced concrete dams. Faulty foundations caused the failure. Throughout the greater portion of the dam the cut-off wall was carried down to bed rock. Where the failure occurred, however, the cut-off wall was carried down only to hard-pan which was 10 ft. above the surface of the bed rock. A narrow band of sand existed only a few inches below the cut-off wall in this section, and through this layer of sand, which was intermixed with clay and some coal, the impounded water seeped out and gradually gaining in velocity of flow and transporting power undermined enough of the dam to induce failure by flexure.

The usual exploratory pits, in this case 6 to 8 ft. square, were sunk to test the nature of the natural foundation for the dam, but these pits, in the section where the cut-off wall was not carried down to bed rock, were only three in number. These pits ranged in depth from 18 to 22 ft., it is reported, and from the bottoms of these test pits coal-auger borings were made to an additional depth of 14 ft. Two of these three auger borings reached bed-rock; the third, the one closest to the end of the dam, did not. The test pit closest to the section which ultimately failed was 200 ft. from it. All test pits were sunk along the crest line of the dam, which is several feet from the cut-off wall. In the full length of the dam, 1,065 ft., only 20 of these exploratory test pits were put down.

In comparing these two dam failures there are two points of strong similarity, namely, each failure resulted from defective foundations, and in both cases the construction proper was well executed.

There are several points of marked dissimilarity of which the following are mentioned: The Austin designer was instructed to keep the cost down to a minimum and in designing he made three separate designs, reducing each cross section a little until he got the least which he believed to be safe. The owners of the Stony River dam wanted a first class dam above all, and did not impose the minimum cost limitation upon the designer. The collapse of the former dam did not occur until 18 months after its failure, while the wrecked bays of the latter dam collapsed within 24 hours after the first leakage under the cut-off wall was observed. While both failures resulted from defective foundations the fault in one case was in not understanding the tendency of one rock stratum to slide upon another, and in the other case the fault was in failing to carry the cut-off wall to rock.

Briefly, the lesson which we would have engineers draw from the failure of the Stony River dam is the same as that indicated in our editorial of Dec. 11, 1912, to which we have referred. The state control of the design, construction and operation of dams is desirable when viewed from the standpoint of the engineer, the owner and the public. The designer of the Austin dam said its failure was not the result of poor workmanship, but of poor judgment upon his part in determining upon its foundation. He stated that he should have sought the advice of a man more skilled than himself in determining foundations for dams. He stated that had there been a state censor of dam engineering it might have resulted in saving this dam. We believe that such an officer would very likely have suggested a proper method of safeguarding the design of this dam and would have done so independently of the owner's insistance that construction costs be kept down. The Austin dam affords a striking illustration of the need of state supervision of dam operation since the broken concrete wall was allowed to serve as a dam for a year and a half before it collapsed. It would have been a poor state officer indeed who would have tolerated such a practice.

Similarly we believe that a state censor of dams would have insisted that the cut-off wall of the Stony River dam be carried down another 10 ft. to bed rock to make a proper water seal below the dam. Such action would probably have saved this partial failure and would have made impossible the exaggerated newspaper accounts of the failure which unquestionably has tended to still further weaken popular confidence in the stability of all dams. That the failure was not unthought of is indicated by the fact, as we have it on good authority, that the contractors who took over the work from the original contractors after one-fourth of the dam had been constructed pointed out in writing, before starting work, the possibility of a failure at just the point where the failure occurred. We consider it extremely probable that a state censor of dams would have heeded this advice and would have insisted upon proper cut-off wall construction.

Professional Standing and Pay of Engineers.

That engineers and engineering societies are giving increased attention to those matters which affect their professional standing and material welfare is evident. It is undoubtedly true that engineers can and should exert a wider influence in public affairs than they now do. Members of the engineering profession are revising their ideas as to what constitutes engineering practice and are broadening their fields of endeavor. In practically all discussions which arise the term "professional dignity" is an important factor. If it is true that engineers are poorly paid, when the service rendered by them is taken into consideration, then we do not believe that their professional dignity will suffer from legitimate endeavors to increase their stipend. The recent address by the president of the Illinois Society of Engineers and Surveyors on "The Functions of the Engineering Society," which appears in this issue, emphasizes the fact that this society, at least, is actively engaged in an attempt to promote the material welfare of its members and of engineers in general. In this address the writer points out some things which have been accomplished, and contends that a proper function of the engineering society consists of actively promoting the material status of the engineering profession and of its individual members. Other organizations are engaged in similar work, more or less actively, and the whole question is one that requires a solution. If the present professional standing and pay of the members of the engineering profession are satisfactory, then the problem will naturally work itself out; but the increasing agitation of the question rather implies that satisfactory conditions do not exist and that concerted action on the part of engineers and engineering bodies is needed.

Roads and Streets

Method and Cost of Constructing Concrete Road With Bituminous Wearing Surface in California.

Contributed by C. L. Rakestraw, Resident Engineer, California Highway Commission.

This article is compiled from data accumulated in constructing 14 miles of state highway (Contract No. 16) (Fig. 1) from Healdsburg to Santa Rosa, California.

SURVEYING.

The salient feature of the survey methods was the combination into one operation of the preliminary surveys and the location. This procedure reduced the number of men required for surveys and made it unnecessary to go over the same ground twice. It also eliminated an office force to study and work up the location. The construction survey was very accurately made, first because of the policy of the Commission to maintain a high standard of accuracy and second because exceptional accuracy was demanded by the probable increase in land values which would result from the improvement. The angles in all transit points, or I. T.'s, were read as azimuth angles and doubled. At each angle point a suitable easy radius curve (minimum radius [illegible] ft.) was run on the ground and the stationing of the survey carried continuous around the curve. The angle points were always referenced out either by the "cross-reference" or the "three-way" tie method to telephone poles, power poles, or trees. The following procedure was followed unless the country was so rough that scout levels and preliminary angle lines were necessary:

The chief of party went ahead and decided upon the location of the center line, considering right-of-way, grades and alinement. The transit men followed, laying out the line. At every "station," "half station" (+50) and curve point the line was marked by driving a 3x$\frac{3}{8}$-in. track spike through four thicknesses of red bunting. These points were temporary, and were used in taking topography of fences, poles, houses, culverts, etc. After the center line was run, the party would place "construction stakes," one on each side of the center line at each "station" and "half station," and at right angles to the center line. These stakes were set far enough to the side of the center line to prevent their disturbance by the construction force. The stakes were marked, proceeding with the survey from zero station, as follows: The stake on right-hand side opposite each point was marked on the side, facing Station 0 with its station number, and on the opposite side with its distance out from the center line; the stake on the left-hand side opposite each point was marked on the side toward Station 0 with its distance out from the center line, and on the opposite side with its station number.

Cross sections were next taken in sufficient detail to show all data necessary to calculate yardages; special refinement was employed in determining cross sections at streams and other places, where a new struc-

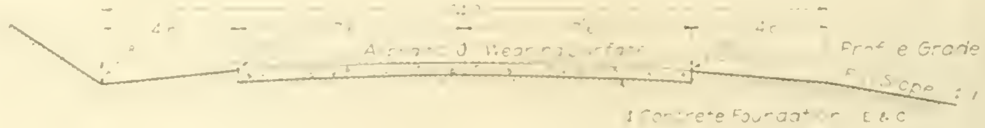


Fig. 1. Cross Section of California State Highway from Healdsburg to Santa Rosa.

ture was required or revetting was needed. Accurate notes were taken on all existing structures; these were in enough detail to permit recalculation of the structure to determine its adequacy under the adopted 15-ton load of the Commission. When taking cross-sections the ground elevation at each construction stake was taken, reading to the nearest tenth of a foot, and the elevation of the top of each stake was taken, reading to the nearest one hundredth foot. These readings were used later as described under the head of rough grading. By this method about 10,000 ft. of line per day was cross-sectioned by a 5-man party. Sections were taken at every station, and half-station, and as close as every 10 ft. at water courses.

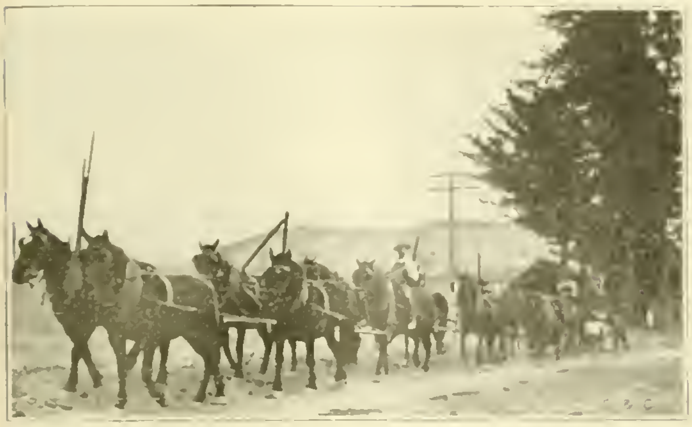


Fig. 2. Rooting Old Roadway with Twelve Head of Horses, California State Highway.

Fig. 4. Placing Side Forms for Concrete Base, California State Highway.

The complete survey usually required one week to complete three miles, and the cost was $315, or about $105 per mile. This cost includes salaries, board and lodging for five men, keep of a team of horses and care of a line wagon. The axe man drove the team. This cost also included the running of a check traverse which had to close 1 in 5,000, and a set of check levels over the benches, which were set every 1,000 ft. along the line.

BREAKING UP OLD ROADS.

Most of the construction was over old roads, and the breaking up of the old road surface was necessary. These old surfaces were often quite hard, due to years of compacting by traffic. When old macadam is 6 to 10 ins. thick and has been compacted from three to five years, it is practically impossible to break up the crust by teams and plows. First, the line of teams (16 to 20 horses) will be so long that three or four drivers are necessary, and it is much trouble to turn the line around. Second, the power from this line of teams will be so unsteady that it is very hard for the four plow men to guide the rooter or plow. The following, based an actual work, is the cost of hard rooting, using teams:

Item—	Cost per day.
20 head, rent with harness, at 50 cts	$10.00
20 head, feed and lodging, at 50 cts	10.00
4 teamsters, at $3.25	13.00
4 plow men, at $2.75	11.00
Depreciation on equipment	0.32
Sharpening 10 plow points, at 10 cts	[illegible]
Supervision supt $1 and foreman [illegible] cts	1 [illegible]
Total	$46.62

Fig. 5. Preliminary Preparation of Subgrade, California State Highway.

This outfit would root about 1,000 ft. per day, and the cost per lineal foot was therefore about 4.7 cts. Compared with this, an 15-ton Kelly Springfield road roller would, with the same rooter and points, root with only three plow men, root 1,500 ft. per day at the following cost:

Item—	Cost per day.
Roller, including depreciation	$1[illegible]
Engineer	4 [illegible]0
Fuel, oil, grease	2.35
Depreciation, plow and points	0.82
Sharpening 13 plow points, at 10 cts	1.30
Supervision, supt., $1, and foreman, [illegible]	1.[illegible]0
3 plow men, at $2.75	8.25
Total	$27.72
Total per lineal foot	1.85 cts.

Comparison of the two statements shows in favor of road roller rooting a saving of 2.83 cts. per lineal foot, or $149.42 per mile of road. These figures are for hard rooting. When a roadway has only about 2 ins. of macadam on the surface it can easily be rooted with twelve head of horses (Fig. 2) and a road plow or rooter at a very reasonable cost, as shown by the following statement:

Item—	Cost per day.
12 head rent with harness, at 50 cts	$ 6.[illegible]
12 head feed and lodging, at 50 cts	6.0[illegible]
2 teamsters, at $3.25	[illegible]
3 plow men at $2.75	8.25
Depreciation on plow, points, etc	[illegible]
Sharpening 15 points at 10 cts	[illegible]
Supervision, supt., $1, and foreman [illegible]	[illegible]
Total	$[illegible]
Total per lineal foot	[illegible] cts.

This outfit will plow 1,500 ft. per day and the unit cost given above is based on this output of work. For this reason, rooting by horses is the cheapest method and it has the additional advantage that the travel of the horses breaks up the clods from the rooter. When a rooter is used for rooting it is generally the practice to run a 6-horse plow back and forth through the material

Fig. 3 Rough Grade Before Staking for Side Forms, California State Highway.

Fig. 6. Checking Subgrade Before Material is Distributed for Mixer, California State Highway.

until all the larger lumps are broken, and it is in shape for the road graders and fresnoes to handle readily. The difference in hardness of the macadam will not affect the output of the steam roller, because the roller has a fixed rate of speed, and can accomplish only the fixed amount of rooting, whether in a soft or hard material.

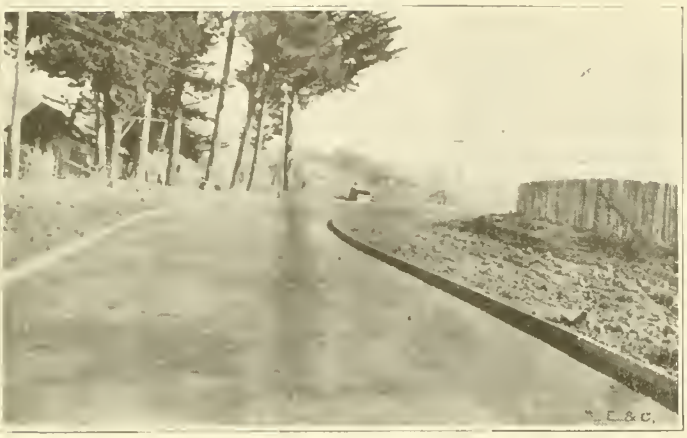


Fig. 7. Finished Subgrade Showing Superelevation on a 300-ft. Radius Curve, California State Highway.

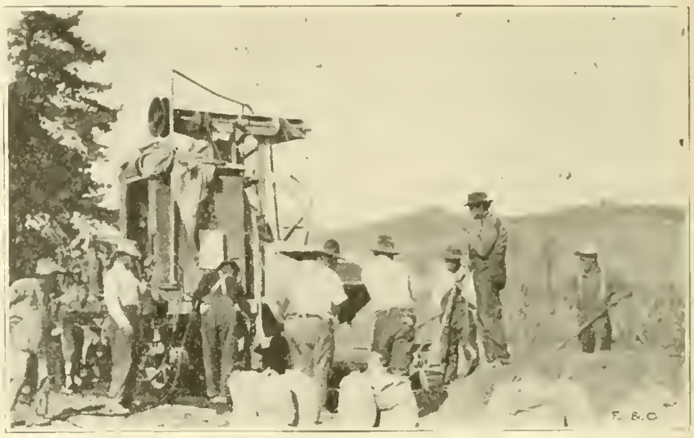


Fig. 8. Distribution of Material for Mixer Gang, California State Highway.

ROUGH GRADING.

After breaking up the old road surface, it was rough-graded (Fig. 3) to within 0.1 ft. of the required grade when rolled for the whole width of the road, including the shoulders. If the shoulders are graded at this time the material is handled with fewer teams than if the grading is done after the concrete is laid. Also the constant compacting of the shoulders by work teams during the concreting puts them in better shape, and ensures a more uniform appearance. At this point the use of the construction stakes described under Surveying came in.

The construction stakes at the time of the location were placed opposite each station and each half station and far enough to one side not to be liable to be disturbed. During cross-sectioning the elevation to the nearest one-hundredth foot of each stake top was determined. As the adopted profile of the road was shown on the plans, to determine the cut or fill on center line it was only necessary to make a simple computation, using the profile grade elevation and the elevation of the top of the stake. This cut or fill was then marked on the side of the stake which faced the center line; one of the assistant resident engineers did this marking, and he could mark as many as three miles of stakes in a day. With the stakes marked, the construction foreman could, with a level rod and hand level, determine the elevation of rough grade at center line. It should be noted here perhaps that the assistant resident engineer in marking the stakes must take into account the thickness of the pavement; for example, in the Healdsburg-Santa Rosa road the concrete pavement was 4 ins. (0.33 ft.) thick, with a crown of 2½ ins. (0.21 ft.), and the rough grade line was 0.33+0.21= 0.54 ft. below profile elevation, and this corrected elevation was the elevation indicated by the figures on the stake. In summary, this system of staking does away with all need of disturbing the stakes during construction; it enables the superintendent of construction to check his rough grade elevation as frequently as he desires, and it makes unnecessary the frequent presence of an engineer to give grades.

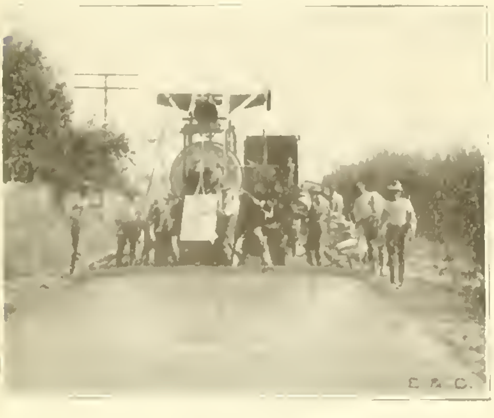


Fig. 9. Concreting Roadway, California State Highway.

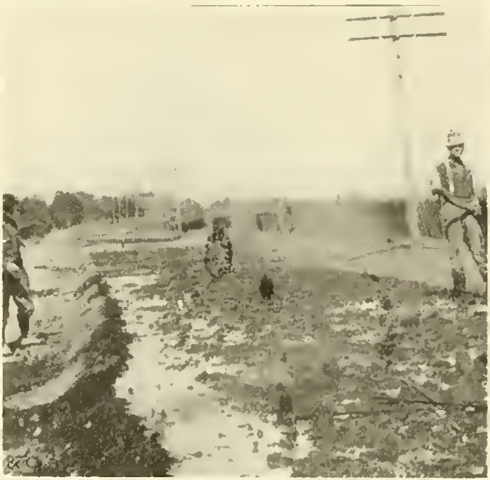


Fig. 10. Curing Concrete with Soaked Layer of Earth, California State Highway.

STAKING AND PLACING SIDE FORMS.

After the rough grade had been brought to within 0.1 ft. of exact elevation, the grade and line stakes were set. The road being particularly described had a 15-ft. pavement with a 4½-ft. shoulder on each side. The section Fig. 1 gives other dimensions. The concrete base has a crown of 2½ ins., the arc of the crowning being a parabola. From the edges of the concrete slab the shoulders slope straight, dropping 4 ins. in 4½ ft.

The location of the side forms (Fig. 4) was 7½ ft. from the crown, and with their top edges 2½ ins., or 0.21 ft., lower than the crown elevation shown in the profile. Two lines of grade stakes were employed, a line 9½ ft. each side of the center line and 2 ft. out on each side from the side forms for the concrete roadway. These 2-ft. intervals gave ample room to set and peg the side forms without disturbing the grade stakes; also the carpenters in setting the side forms with a 30-in. spirit level could notch the level 2 ft. from the end, and with it alone adjust the form board both to distance and grade, working from the grade stakes. The carpenter setting side forms required only a spirit level and a hammer.

The side forms were handled usually by two men at $2.75 per day and two men at $2.50 per day. These could place, line up and fasten about 800 ft. on each side, or 1,600 ft. of side forms per day at the following costs:

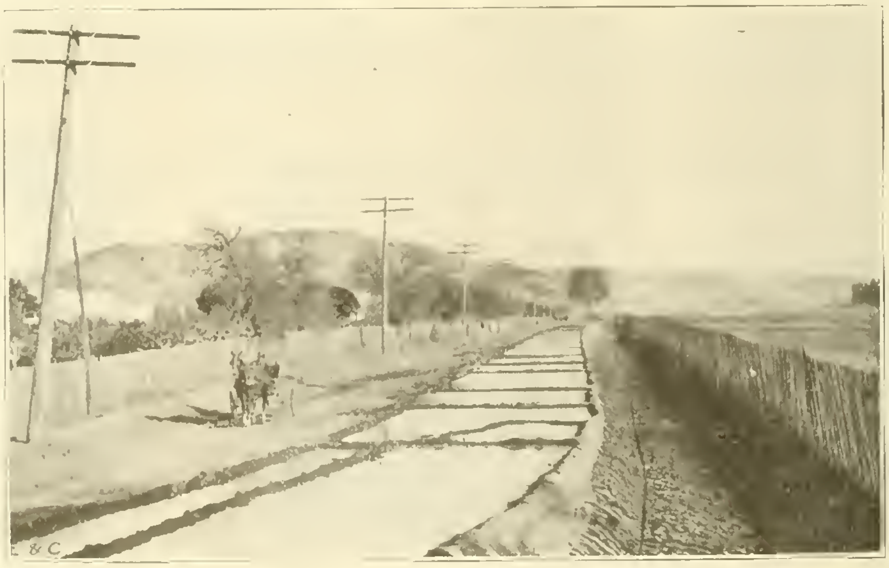


Fig. 11. Levee and Basin Method of Curing Concrete, California State Highway.

Item—	Cost per day.
2 carpenters at $2.75	$ 5.50
2 helpers at $2.50	5.00
*1,600 ft. 2x4-in. plank at $26 per M ft. B. M.	9.27
Nails, etc.	0.23
200 stakes at 1 ct	2.00
Interest and depreciation	0.15
Superintendent, $1; foreman, 50 cts	1.50
Total for 800 ft. of road	$23.65
Per lineal foot of road	2.96 cts.

*Boards used three times, so one third of total cost is charged.

All curves were superelevated so a vehicle traveling at legal speed of 20 miles per hour would ride easily. For staking curves Table I of superelevations for a 15-ft. roadway was computed by the author and employed with satisfactory results. To use this table one must remember to maintain the inside form board at grade and to superelevate the outside form only. In some cases on short curves it was found that superelevation if started at zero on the P. C. and increased to maximum at curve center was unsightly. In such cases the superelevation was begun one-half the actual curve length before P. C., and ended the same distance beyond P. T.; the superelevation was then one-half at P. C., maximum at curve center and one-half at P. T.

SUBGRADE CONSTRUCTION.

The side forms being placed and checked for accuracy, material was filled between them and spread over the rough grade in proper depths to give the correct cross-sectional profile. (Fig. 5). This filling was dampened to the proper consistency to roll solidly. Rolling was continued until the soil failed to yield, i. e., to creep, roll or crack, under the roller, and was so hard that it could not be dented with the heel of one's shoe. This rolling and wetting was the most difficult task of the work, and it was found that too much care cannot be taken, especially at points over small pipes and culverts and near the back fills of structures.

The proper amount of water to add to various soils requires good judgment, and if misjudged often causes delay to the concrete laying because of unreadiness of the subgrade. A soil with much sand and gravel and good drainage can stand a good wetting and be rolled soon afterward, but clay and adobe present more difficult problems.

With clay it was found, after numerous trials with and without wetting, best to leave it as found in the ground (unless wet and boggy, when it should be removed), and add to it a damp, coarse sand or a gravel. The damp gravel adds all the moisture needed to make the clay-gravel mixture work easily, and also prevents the mixture from clinging to the roller.

With adobe the conditions are entirely different, as shown by repeated experiments by the writer on the work being described. If wetted the water simply "glazed" the surface and made it slippery. If no water was added the adobe would slide away from the roller and "roll up" in front of it. If sand or gravel were added it would be pushed down into the adobe and stay there with no effort whatever. The following method was finally worked out as the best: Excavate to several inches below grade between the side forms, dampen the surface and sprinkle on about 1 in. of dry adobe and then roll. After this layer was compacted, dampen its surface and sprinkle on another layer of dry adobe and roll it. Repeat these operations until the fill was brought to grade and profile. By this method the adobe keeps proper consistency and will not roll up ahead of the roller or shift or slide.

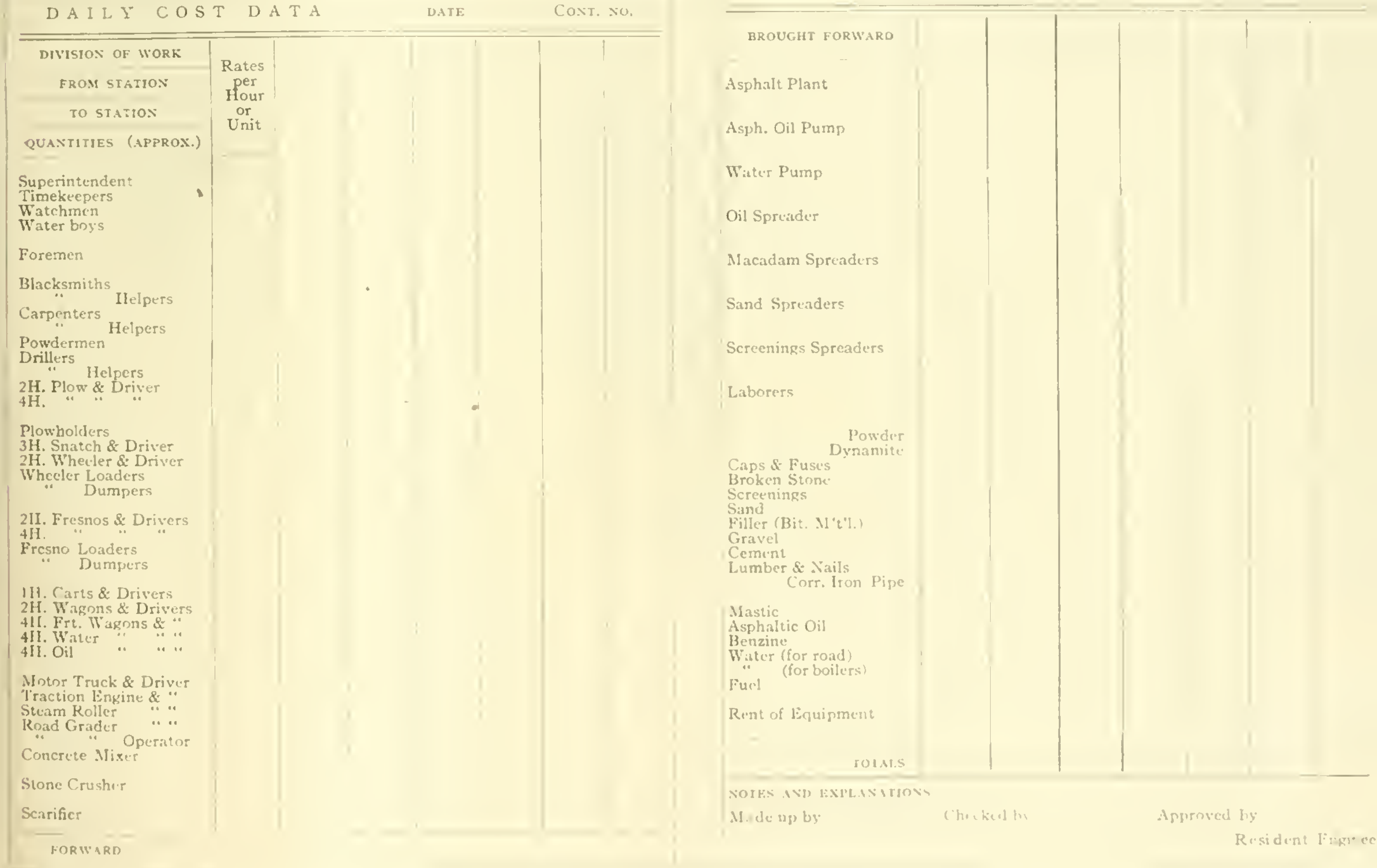
Another method of handling adobe subgrade for concrete road where sand is plentiful and cheap is to follow the practice in cement walk construction on adobe. Here the adobe is finished off to a level about 1 in. below grade, and a 1-in. layer of sand is spread over it. On this sand cushion the cement slab is built. This sand cushion appears to act as a sort of expansion joint, which takes care of the swelling and shrinking of the adobe, due to wetting and drying and prevents dangerous movement of the sidewalk slab. This method requires a large amount of sand, and unless sand is very cheap the rolling method first described is much cheaper and practically as satisfactory.

When the first rolling of the subgrade had been completed the resident engineer or his inspector checked its surface by means of the templet shown by Fig. 6. If the templet showed variations of consequence from true grade, the surface was harrowed and material added or removed in such amounts as would give the true surface and grade when rerolled. If lack of compactness, such as followed oil dripping from the roller, was exhibited in spots, these spots were dug up and new material was added and hand tampe until hard. Attention is called to the handle on the templet as shown in Fig. 6; the were found very convenient, and they en abled four men to reshape a large amoun of subgrade in a short time.

The cost of reshaping subgrade (Fig. 7 varies so much with the material that it tabulation by items is impossible. It runs however, about ¾ ct. per square foot for 15-ft. pavement; this cost is exclusive o grading, excavation and fill. Excavation where it consists of shaving off a thin laye of crust here and there, costs about 80 cts to 90 cts. per cubic yard.

MIXING AND LAYING CONCRETE.

The specifications called for a 1:2:4 broke stone concrete and stated the sizes of ston and sand. With the undertaking of work i was found that a natural gravel pit existed a Healdsburg and that the company operatin



DAILY COST DATA DATE CONT. NO.

DIVISION OF WORK
FROM STATION
TO STATION
QUANTITIES (APPROX.)

Rates per Hour or Unit

Superintendent
Timekeepers
Watchmen
Water boys

Foremen

Blacksmiths
" Helpers
Carpenters
" Helpers
Powdermen
Drillers
" Helpers
2H. Plow & Driver
4H. " " "

Plowholders
3H. Snatch & Driver
2H. Wheeler & Driver
Wheeler Loaders
" Dumpers

2H. Fresnos & Drivers
4H. " " "
Fresno Loaders
" Dumpers

1H. Carts & Drivers
2H. Wagons & Drivers
4H. Frt. Wagons & "
4H. Water " " "
4H. Oil " " "

Motor Truck & Driver
Traction Engine & "
Steam Roller " "
Road Grader " "
" " Operator
Concrete Mixer

Stone Crusher

Scarifier

FORWARD

BROUGHT FORWARD

Asphalt Plant

Asph. Oil Pump

Water Pump

Oil Spreader

Macadam Spreaders

Sand Spreaders

Screenings Spreaders

Laborers

Powder
Dynamite
Caps & Fuses
Broken Stone
Screenings
Sand
Filler (Bit. M't'l.)
Gravel
Cement
Lumber & Nails
Corr. Iron Pipe

Mastic
Asphaltic Oil
Benzine
Water (for road)
" (for boilers)
Fuel

Rent of Equipment

TOTALS

NOTES AND EXPLANATIONS

Made up by Checked by Approved by

Resident Engr ee

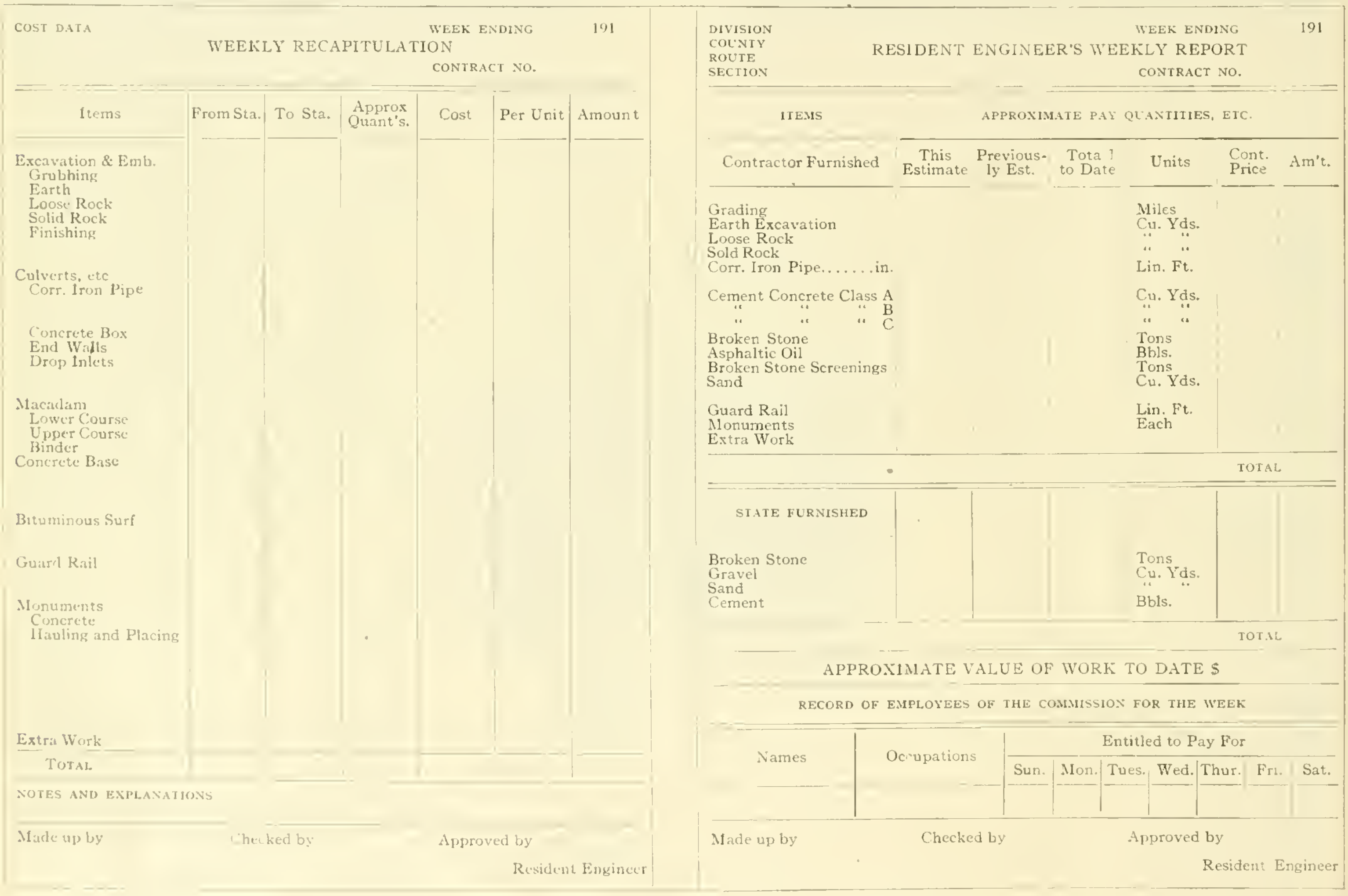
Fig. 12. Daily Cost Report, California State Highway.

(Ruled sheet, $8\frac{1}{2}$x11 ins., Items continued on reverse as indicated.)

this pit had facilities for furnishing promptly this gravel in any quantity demanded by the work. The gravel was exceptionally clean washed gravel, well graded and dense. Gravel was, therefore, substituted for broken stone. It cost delivered to any railway siding along the work 72 cts. per cubic yard. The average haul from railway to the work was 1½ miles, and the cost of hauling was 63 cts. per cubic yard, or 41 cts. per yard mile. This cost of haul seems high, but it is accounted for by the weight of the gravel, 3,300 lbs. per cubic yard, and by the fact that crooked roads prevented haulage in wagon train by traction engines and made team hauling necessary.

Using this gravel and a 1:6 mixture, it was determined that 96 sacks of cement would make 100 ft. of 15 ft. by 4 in. pavement. The plan adopted was to pile the gravel continuously along the middle of the subgrade and place the cement in four-sack piles spaced 4 ft. apart (Fig. 8). A cleat was riveted to the inside of the mixer charging hopper to indicate a two-sack batch of 1:6 mix. Six men using square-pointed shovels charged the gravel and one man charged the cement. Golden Gate cement in cloth bags was used; paper sacks broke easily and carried water, and also the fog loosened the paste, letting the sacks open and admit dampness.

The mixer traveled on 3 ft. x 3 in. redwood sills, which could be shifted easily and often enough to guide the mixer well. This runway was located midway between side forms, which shifted the discharge chute slightly off center, but not enough to inconvenience the concreters. As the mixer was operated some 1½ days behind the subgrade finishing crew, the subgrade surface had opportunity to dry out, and consequently it was wetted down ahead of the concrete laying, so that moisture would not be sucked by the soil from the concrete.



COST DATA					WEEK ENDING	191
WEEKLY RECAPITULATION						
					CONTRACT NO.	
Items	From Sta.	To Sta.	Approx Quant's.	Cost	Per Unit	Amount
Excavation & Emb.						
Grubbing						
Earth						
Loose Rock						
Solid Rock						
Finishing						
Culverts, etc						
Corr. Iron Pipe						
Concrete Box						
End Walls						
Drop Inlets						
Macadam						
Lower Course						
Upper Course						
Binder						
Concrete Base						
Bituminous Surf						
Guard Rail						
Monuments						
Concrete						
Hauling and Placing						
Extra Work						
TOTAL						

NOTES AND EXPLANATIONS

Made up by Checked by Approved by

Resident Engineer

DIVISION
COUNTY
ROUTE
SECTION

RESIDENT ENGINEER'S WEEKLY REPORT

WEEK ENDING 191

CONTRACT NO.

ITEMS	APPROXIMATE PAY QUANTITIES, ETC.					
Contractor Furnished	This Estimate	Previously Est.	Total to Date	Units	Cont. Price	Am't.
Grading				Miles		
Earth Excavation				Cu. Yds.		
Loose Rock				" "		
Sold Rock				" "		
Corr. Iron Pipe.......in.				Lin. Ft.		
Cement Concrete Class A				Cu. Yds.		
" " " B				" "		
" " " C				" "		
Broken Stone				Tons		
Asphaltic Oil				Bbls.		
Broken Stone Screenings				Tons		
Sand				Cu. Yds.		
Guard Rail				Lin. Ft.		
Monuments				Each		
Extra Work						
					TOTAL	
STATE FURNISHED						
Broken Stone				Tons		
Gravel				Cu. Yds.		
Sand				" "		
Cement				Bbls.		
					TOTAL	

APPROXIMATE VALUE OF WORK TO DATE $

RECORD OF EMPLOYEES OF THE COMMISSION FOR THE WEEK

Names	Occupations	Entitled to Pay For						
		Sun.	Mon.	Tues.	Wed.	Thur.	Fri.	Sat.

Made up by Checked by Approved by

Resident Engineer

Fig. 13. Weekly Cost Report, California State Highway.

(Facing sheets each 8½x11 ins.)

The concrete was distributed by the chute (Fig. 9), and also shoveled against the side forms, special care being taken to well spade and dump the concrete against the forms, so as to ensure an exceptionally dense and strong concrete next the shoulders, where severest wear comes.

At this point note may be made with reference to expansion joints. Though many engineers and paving contractors insist that concrete pavement must have expansion joints, none were used in this work. The writer believes them to be unnecessary in concrete pavement, for the reasons to be given, and the results with the work being described confirm his belief. Expansion joints require placing cross-forms and stopping the mixer gang every 16 to 20 ft. until the felt or fiber joint filling is in position. There is of course the other method of cutting the joint in the soft concrete with a steel edge and then inserting the joint filling, but this is a sloppy method and usually mars the pavement surface. Now, what is the use of placing expansion joints at stated intervals? Why go to the expense and labor for this operation when nature will perform it better than any human means? Suppose we lay a strip of concrete pavement 15 ft. wide, 4 ins. thick and 100 ft. long without expansion joints; we all know that this pavement will not remain in one strip, but will crack at points about 25 ft. apart, and, furthermore, these cracks will be at the weakest points in the slab. If we were placing expansion joints in this stripe of pavement we would probably have five joints, and these joints probably (almost positively) not at the weakest places in the pavement. Therefore by leaving out the expansion joints and letting the expansion of the pavement itself break the pavement, we have the maximum of this pavement in the largest possible slabs. Now, after cleaning the concrete slab for the application of the wearing surface, specially clean these cracks and pour hot or heavy asphaltic road oil into them; this will form a perfect expansion joint.

The cost of the concrete base laid as outlined above was as follows:

Item—	Cost per day
1 foreman at $4	$ 4.00
1 engineman at $3	3.00
10 shovelers at $2.75	27.50
1 cement man at $2.75	2.75
2 finishers at $3	6.00
Depreciation of plant and tools	9.00
Cost of water	6.13
Total	$57.38

The average daily run of concrete pavement was 550 lin. ft., or 101.85 cu. yds. The above statement of costs is a statement of costs with concrete materials—cement and gravel—delivered onto the subgrade ready for use, and they give the following unit costs:

Per lin. ft., 15-ft. roadway	10.4 cts.
Per cu. yd. of concrete	56.3 cts.

METHOD AND COST OF SECURING WATER.

All water used on this work had to be pumped. A line of 2-in. boiler pipe in 8-ft. lengths was laid along the work and provided with 1-in. taps at 50-ft. intervals. This pipe line was torn up and laid ahead as pumping stations were moved and as the work progressed; two men at a daily cost of $5 were employed continuously at this work. All pumping stations but one were located at adjacent streams, and the water supply cost nothing except for pumps and pumping. In one case an 180-ft. well was bored at a cost of $380; this well supplied 100,000 gals. per day during the driest part of the year for about five miles of the road. The cost per day of supplying water was as follows:

Item—	Cost per day.
1 gas engine pump man at $2.75	$ 2.75
2 gas engine pump men at $2.50	5.00

2 pipe layers at $2.50	5.00
Int. and dep. on 17,000 ft. of pipe and engine	2.00
Fuel and oil on gasoline pump	2.15
Superintendent, $1; foreman, 50 cts	1.50
Total	$18.40

Water was used in about equal quantities for (1) mixing concrete; (2) curing concrete and (3) wetting subgrade; of the above total cost, therefore, one-third or $6.13, was charged to each service.

FINISHING AND CURING.

Several methods of finishing and curing the concrete slab were investigated. The first plan was: Six hours after placing to broom with a steel broom the surface and so roughen it that the bituminous covering would cling. After twelve hours sprinkling began and the concrete was kept moist. While this plan might have been satisfactory in winter or wet weather it did not give good results in the summer which was the time of year the work was done. The concrete could not be kept evenly moist and also much water ran off and was wasted. The second plan (Fig. 10) was to broom the concrete as in the first plan, then build earth dams along the pavement edges, then wet down the concrete, then cover it with 2 ins. of earth and water the covering until saturated and the water showed in pools. By this plan the moisture was better distributed, it was more obvious to common laborers when more wetting was needed, and there was less loss of water by evaporation and run off. A third plan tried and abandoned was, after brooming the concrete, to sprinkle it and cover it with heavy building paper held down by clods and stones. The idea was to remove the paper, resprinkle and replace the paper every night for seven nights, the standard curing period. It was anticipated that the paper would prevent evaporation, reduce labor as compared with shoveling in earth and building dams, be quick in application, eliminate attention during the day and offer several other advantages. The plan did reduce labor but the paper was torn off in places by the wind and did not protect the pavement from drying out in spots. It was no better than the first plan.

A fourth plan (Fig. 11) was finally devised which eliminated most of the faults of preceding plans. First, levees were built along the edges and over the side forms in such position that about one-third the width of the embankment fell inside the form board and over the concrete. These side levees were built high enough to hold a depth of water of 2 ins. over the crown of the slab. At suitable intervals depending upon the grade, cross levees connecting the side levees were built. These levees divided the pavement into a series of basins which could be filled with water as indicated by Fig. 11. On super-elevated curves in addition to cross levees a number of parallel longitudinal levees were built.

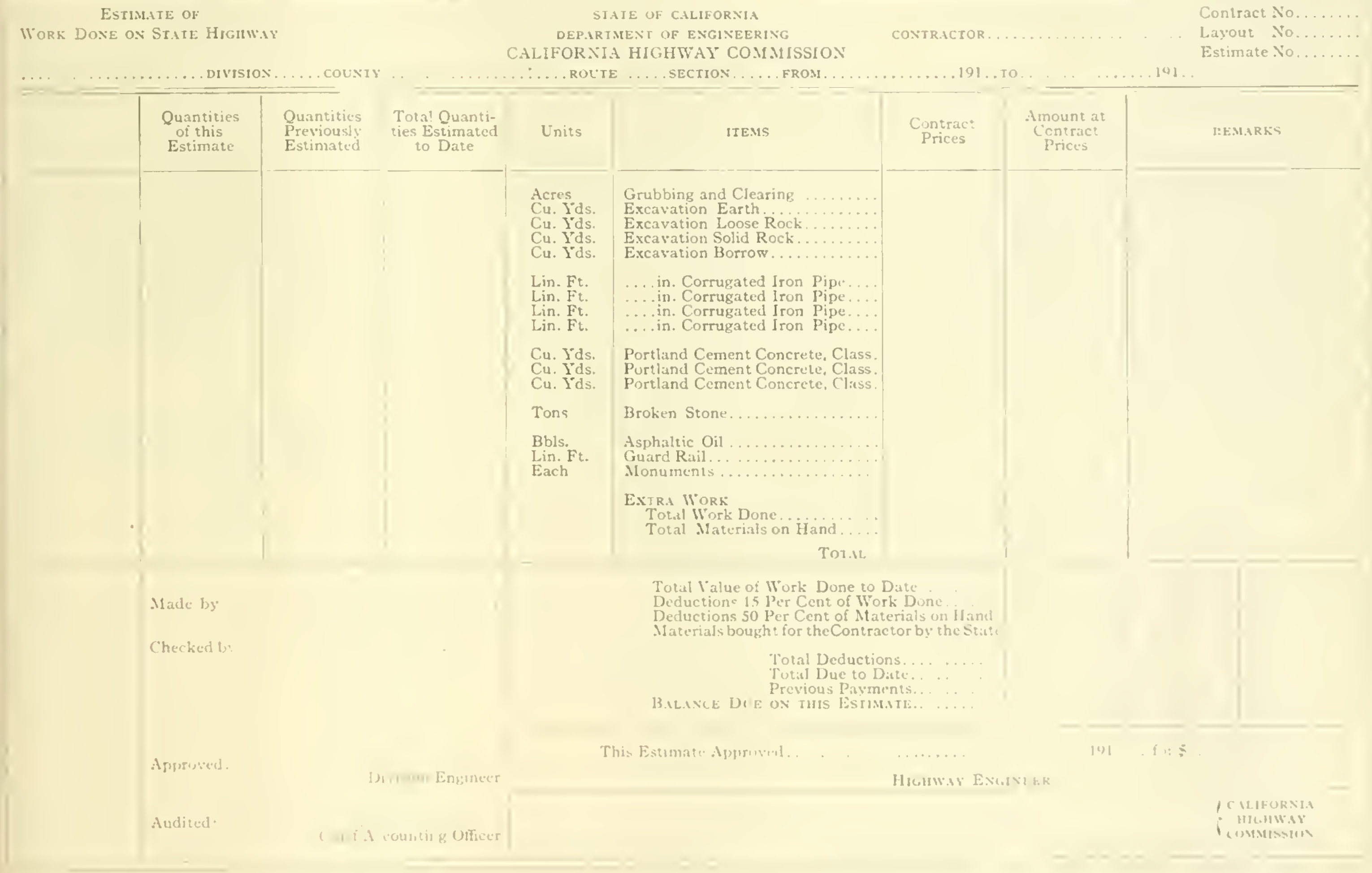
Referring to some of the details noted above. The purpose of building the side levees two-thirds outside the side forms is two-fold: First, about one-third the width of the levee becomes saturated and this third is over the concrete slab which requires wetting. Second, the form boards can be lifted out for reuse leaving two-thirds of the levee intact to maintain the reservoir. Besides being required for hydraulic reasons the division by cross levees into small basins serves the purpose of confining loss of water by a levee break to a small area of pavement; restoration is also thus facilitated. Also in construction the workman can let one basin be filling while he is building the succeeding levees.

This method of watering concrete pavement had the following advantages over the second described and next most successful method: (1) It required less labor to construct cross levees than to cover the slab all over with 2 ins. of earth; (2) the wetted black earth covering suffers greater loss by evaporation than does the heat reflecting water surface; (3) all the pavement is water covered while an earth covering may dry out in spots and absorb water from the concrete; (4) all work is done at night when water is needed for no other purpose, while an earth covering has to be sprinkled continuously; (5) one filling of the basins suffices for the total curing while an earth covering has to be wetted frequently; (6) the levees suffice as barriers notifying drivers not to cross the work, while with earth covering separate barriers are necessary; (7) the filling of the basins, however, must be more carefully done so as not to wash the concrete than when earth covering is used; the best method is

TABLE I — SUPER - ELEVATIONS FOR CURVES ON CONCRETE ROAD IN CALIFORNIA.

(Roadway 15 ft. wide.)

Radius of curve	Super-elevation.	Radius of curve.	Super-elevation.
300	.79	1,400	.17
350	.69	1,450	.17
400	.59	1,500	.16
450	.53	1,550	.16
500	.48	1,600	.15
550	.44	1,650	.15
600	.40	1,700	.14
650	.37	1,750	.14
700	.35	1,800	.13
750	.33	1,850	.13
800	.30	1,900	.12
850	.28	1,950	.12
900	.26	2,000	.12
950	.25	2,500	.10
1,000	.24	3,000	.08
1,050	.23	3,500	.07
1,100	.22	4,000	.06
1,150	.21	4,500	.06
1,200	.20	5,000	.05
1,250	.19	5,500	.05
1,300	.18	6,000	.04
1,350	.18	6,500	.04



ESTIMATE OF
WORK DONE ON STATE HIGHWAY

STATE OF CALIFORNIA
DEPARTMENT OF ENGINEERING
CALIFORNIA HIGHWAY COMMISSION

CONTRACTOR……………………

Contract No……..
Layout No……..
Estimate No……..

……………………DIVISION……COUNTY……………ROUTE……SECTION……FROM……………191..TO……………191..

Quantities of this Estimate	Quantities Previously Estimated	Total Quantities Estimated to Date	Units	ITEMS	Contract Prices	Amount at Contract Prices	REMARKS
			Acres	Grubbing and Clearing			
			Cu. Yds.	Excavation Earth			
			Cu. Yds.	Excavation Loose Rock			
			Cu. Yds.	Excavation Solid Rock			
			Cu. Yds.	Excavation Borrow			
			Lin. Ft.	….in. Corrugated Iron Pipe			
			Lin. Ft.	….in. Corrugated Iron Pipe			
			Lin. Ft.	….in. Corrugated Iron Pipe			
			Lin. Ft.	….in. Corrugated Iron Pipe			
			Cu. Yds.	Portland Cement Concrete, Class.			
			Cu. Yds.	Portland Cement Concrete, Class.			
			Cu. Yds.	Portland Cement Concrete, Class.			
			Tons	Broken Stone			
			Bbls.	Asphaltic Oil			
			Lin. Ft.	Guard Rail			
			Each	Monuments			
				EXTRA WORK			
				Total Work Done			
				Total Materials on Hand			
				TOTAL			

Made by

Checked by

Total Value of Work Done to Date
Deductions 15 Per Cent of Work Done
Deductions 50 Per Cent of Materials on Hand
Materials bought for the Contractor by the State

Total Deductions
Total Due to Date
Previous Payments
BALANCE DUE ON THIS ESTIMATE

Approved.
Division Engineer

This Estimate Approved…………191 for $

HIGHWAY ENGINEER

Audited:
Chief Accounting Officer

CALIFORNIA HIGHWAY COMMISSION

Fig. 14. Monthly Cost Report, California State Highway.

(Sheet 14x11 ins.)

to let the hose stream run on a sack laid on the pavement.

The cost of curing concrete pavement by the methods described are given by Table II; in this table method three being considered not practical is omitted, also its cost is about the same as that for method one. It is seen from Table II, that method four is far the cheapest.

TABLE II.—COST OF CURING CONCRETE PAVEMENT.

	Method No. 1.	Method No. 2.	Method No. 3.
1 man at $2.75 per day	.....	$ 2.75	$ 2.75
Men at $2.50 per day	$12.50	17.50	10.00
Depreciation, shovels, etc.	0.40	0.80	0.65
Cost of water	6.13	6.13	6.13
Supervision, supt. & fore-foreman	1.50	1.50	1.50
Total cost 1st day	$20.03	$28.68	$20.53
Lineal feet covered	300	550	550
Cost per lin. ft., 1st day	$0.067	$0.052	$0.037
Cost of each consecutive day	0.067	0.052	0.005
Total cost of curing 7 days, per lin. ft. pavem't	0.469	0.364	0.067

The side form boards were removed seven days after placing the concrete; this work cost about 1 ct. per lineal foot of pavement The earth levees were left in place about a week longer and were then removed, usually with a four-horse road scraper and at a cost of about 0.3 ct. per lineal foot of pavement. With the same scraper the shoulders were brought as near as practicable to grade and then they were finished by hand. Constructing shoulders cannot easily be figured in cubic yard units but if they are brought to shape with the rough grading as previously described the work will cost exclusive of rolling, about 5 cts. per lineal foot of pavement and including rolling about 6 cts. per lineal foot of pavement.

SUMMARY OF COST.

Summarizing the costs previously given, the following tabulation is obtained:

	Per ft. road.
Tearing up old roadway with rooter and plows	$0.0283
Placing form-boards (after exc. and emb. have been done)	0.0296
Handling and preparing sub grade with rolling	0.0375
Cost of pouring and finishing 4-in. concrete base	0.1043
Cost of curing and finishing (Method No. 4	0.0670
Cost of removing form-boards	0.0100
Cost of cleaning earth off pavement	0.0030
Cost of preparing shoulders	0.0600
Actual cost per lineal foot	$0.3397
Actual cost per mile	$1,793.62
10 pct. for contingencies	179.36
Total cost per mile	$1,972.98

These costs are exclusive of costs of materials and of excavation and fill.

COST-KEEPING SYSTEM.

In the handling of this work the resident engineer usually had three assistant resident engineers who kept the data and furnished the contractor with all necessary stakes and elevations for bridges, culverts, and alignments, etc. This force cost the state about from 5 per cent to 8 per cent for the engineering work on the contract under the writer and this was considered a fair average.

At the end of each day a "Daily Report" Fig. 12, was made out and the respective quantities were filled in by the several men who were on the work. At the end of each week a "Weekly Report" Fig. 13, was compiled and sent to the Division Headquarters in duplicate. One of the copies was kept at the Division office and the other was sent to Headquarters or to the Highway Engineer. These reports, though concise, gave a very fair recapitulation of what had been done during the week past and one could see whether or not the contractor was making or losing money and just how much.

On the Saturday nearest the 15th of the month the resident engineer was required to make out an estimate of the work done on the contract at the contract prices and segregated under the proper heads. A few peculiarities entered into these estimates due to the fact that the materials were furnished by the state and any waste by the contractor was charged to him and deducted from the amount due at that time. The usual amount of 15 per cent was held back on the contractor at all times as is the regular practice. This report, Fig. 14, was then forwarded to the proper authorities and after being audited the amount was paid to the contractor.

Approximate Stresses Produced by a Concentrated Load on a Continuous Slab Supported on Earth or Other Yielding Foundation.

Contributed by J. W. Pearl, Chicago, Ill.

The following investigation was made to determine the stresses in and thickness of a continuous slab supported on a yielding foundation, due to a concentrated load. Owing to the necessity of making assumptions the stresses obtained by the formulas developed are approximate, but are sufficiently accurate for practical purposes. The discussion is directly applicable to concrete pavement design and is also applicable to the design of flat slabs supported on columns having small diameters.

The pressure developed in the foundation will be very nearly proportional to the deflection of the slab, which is indicated by the curved dotted line in Fig. 1 (a). For all practical purposes the straight line distribution of pressure shown is sufficiently accurate. The intensity of pressure upon the underside of the slab may be very closely represented by a cone with the maximum intensity p under the load P. In the final equations both p and r are eliminated so they may have any reasonable values between limits which would not give excessive deflections or would not cause the earth to flow laterally. In other words, the solution is based upon deflection without rupture and compression without lateral flow.

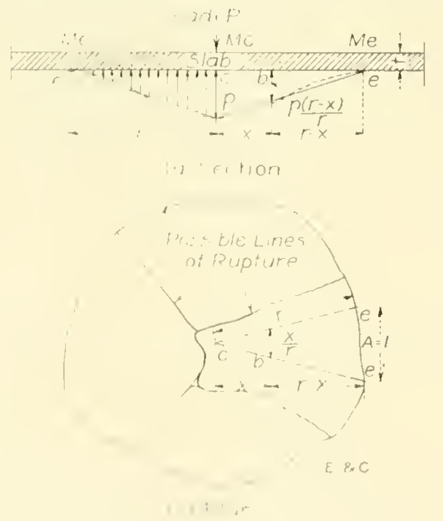


Fig. 1. Section and Plan—Conical Distribution of Pressure Assumed—Yielding Foundation.

As the load is applied a positive moment, M_c, develops at the center and a negative moment, designated by M_e, at a distance r from the center; these moments may be assumed to be nearly equal. If the load is increased to the point of rupture, failure must occur at both c and e, on lines somewhat as represented by the irregular lines in Fig. 1 (b); therefore we will assume in our solution that M_c and M_e are equal, although any other relative value could be used if desired.

From the volume of a cone we have $P = \frac{1}{3}\pi r^2 p$, and for a sector whose arc or chord $= A = 1$, the load will be $\frac{pr}{6} = \frac{P}{2\pi r}$;

whence $p = \frac{3P}{\pi r^2}$ ....(a)

For some radius between M_c and M_e there will be a point of contraflexure distant x from the center, where the intensity of pressure will be $\frac{p(r-x)}{r}$ and the length of arc at this radius will be $\frac{x}{r}$.

The summation of pressures on a sector from the edge at e to the point b will be

$$\frac{p(r-x)}{r} \cdot \frac{x}{r} \cdot \frac{(r-x)}{2} + \tfrac{1}{3} \cdot \frac{p(r-x)}{r} \cdot \left(1 - \frac{x}{r}\right) \frac{(r-x)}{2} = \frac{p(r-x)^2 x}{2r^2} + \frac{p(r-x)^3}{6r^2} \text{ ....(1)}$$

and the summation of pressures from b to the center will be

$$\frac{p(r-x)}{r} \cdot \frac{x}{r} \cdot \frac{x}{2} + \tfrac{1}{3}\left(p - \frac{p(r-x)}{r}\right) \cdot \frac{x}{r} \cdot \frac{x}{2} = \frac{p(r-x)x^2}{2r^2} + \frac{px^3}{6r^2} \text{ ....(2)}$$

Equation (1) also gives the shear at b; therefore the moments will be

$$M_c = \left(\frac{p(r-x)^2 x}{2r^2} + \frac{p(r-x)^3}{6r^2}\right)x + \frac{p(r-x)x^2}{2r^2}\ \tfrac{2}{3}x + \frac{px^3}{6r^2} \cdot \frac{x}{2}$$

$$= \frac{p}{12r^3}(6r^2x^2 - 12rx^3 + 6x^4 + 2r^3x - 6r^2x^2 + 6rx^3 - 2x^4 + 4rx^3 - 4x^4 + x^4)$$

$$= \frac{p}{12r^2}(x^4 - 2rx^3 + 2r^3x) \text{ ....(3)}$$

and

$$M_e = \left(\frac{p(r-x)^2 x}{2r^2} + \frac{p(r-x)^3}{6r^2}\right)(r-x) - \frac{p(r-x)^2 x}{2r^2} \cdot \tfrac{2}{3} \cdot (r-x) - \frac{p(r-x)^3}{6r^2} \cdot \frac{(r-x)}{2}$$

$$= \frac{p}{12r^2}(6r^3x - 18r^2x^2 + 18rx^3 - 6x^4 + 2r^4 - 8r^3x + 12r^2x^2 - 8rx^3 + 2x^4 - 4r^3x + 12r^2x^2 - 12rx^3 + 4x^4 - r^4 + 4r^3x - 6r^2x^2 + 4rx^3 - x^4)$$

$$= \frac{p}{12r^2}(r^4 - 2r^3x + 2rx^3 - x^4) \text{ ....(4)}$$

By adding equations (3) and (4), we have

$$M_c + M_e = \frac{pr^2}{12} \text{ ....(b)}$$

When the moments at the center and edge are equal, $M_c = M_e = \frac{pr^2}{24}$ ....(c)

Engineers will differ regarding the relative value of M_c and M_e, but neither can be as small as zero or greater than $0.083pr^2$, and one or the other will always be equal to or greater than $0.0417\ pr^2$.

Substituting the value of p from equation (a) in equation (c), we have

$$M_c = M_e = 0.0417\ \frac{3P}{\pi} = 0.0398P.$$

Therefore the minimum value of M_c or M_e may be approximately $0.04P$, and the maximum value of M_c or M_e may be approximately $0.08P$.

As these values of M_e are independent of p and r they may truly represent the stress at a fracture at any irregular distance from the point of application of a concentrated load, or upon a regular or irregular radial line.

By expressing x as a ratio of r in equations (3) and (4) and reducing the equations, the coefficients for all possible values of M_c and

M_e are obtained for practical and rapid computation. These coefficients are given in Table I.

Similar values for a case in which the pressure upon the under side of the slab is uniform, as represented in Fig. 2, have been computed, but will not be given. All possible values of the moments per unit width of slab are found independently of the functions p and r, and are platted as curves in Fig. 3.

TABLE I.—COEFFICIENTS FOR USE IN DETERMINING Mc AND Me, ASSUMING CONICAL DISTRIBUTION OF PRESSURE.

Values of $\frac{x}{r}$	0.0	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9	1.0
Mc from Equation (3).											
$Mc = \frac{pr^2}{12}$ multiplied by	0.0	0.1981	0.3856	0.5541	0.6976	0.8125	0.8976	0.9541	0.9856	0.9981	1.00
$Mc = P$ multiplied by	0.0	0.0158	0.0307	0.0441	0.0555	0.0646	0.0714	0.0759	0.0784	0.0794	0.0796
Me from Equation (4).											
$Me = \frac{pr^2}{12}$ multiplied by	1.00	0.8019	0.6144	0.4459	0.3024	0.1875	0.1024	0.0459	0.0144	0.0019	
$Me = P$ multiplied by	0.0796	0.0638	0.0489	0.0355	0.0241	0.0149	0.0081	0.0037	0.0011	0.0002	

In determining the coefficients from Fig. 3 for different values of $x \div r$, any two points on the same horizontal line, at its intersection with the curves representing M_c and M_e, will give the values for the coefficients, for either a uniformly varying pressure or for a uniform pressure. The values of these coefficients are shown at the top of Fig. 3. The moment is then found by multiplying the coefficient by P, the result being in inch-pounds per inch of diameter or periphery or in foot-pounds per foot of diameter or periphery. Intermediate values may be determined with sufficient accuracy by interpolation.

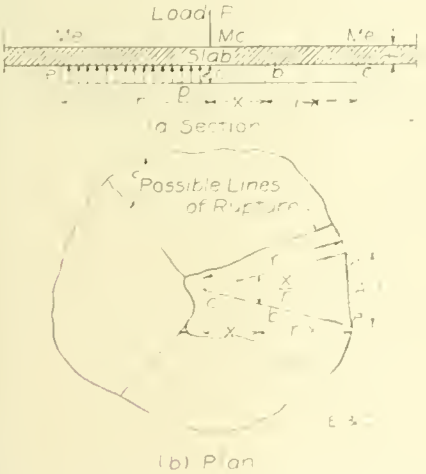


Fig. 2. Section and Plan—Uniform (Cylindrical) Distribution of Pressure Assumed—Yielding Foundation.

The point of contraflexure (zero moment) for the case assumed is given by the value of $x \div r$ at the left end of the horizontal line used.

For the case of a column footing and a non-continuous slab, $M_e = 0$ and $x = r$, and for a conical distribution of pressure $M = 0.0796\,P$.

For any given radius and for a conical distribution of pressure, the summation of M_e will be

$$\Sigma M_e = \frac{p2\pi r}{12r^2}(r^4 - 2r^3x + 2rx^3 - x^4),$$

and when $M_c = M_e$

$$\Sigma M_e = \frac{p\pi r^3}{12}.$$

But $\frac{p\pi r^3}{3} = P \therefore \Sigma M_e = \frac{Pr}{4}$ ....(5)

Writing the moment of resistance for a plain concrete slab for a corresponding radius r, fiber stress f and thickness t, we have

$M_r = \frac{1}{6} f 2\pi r t^2 = \frac{1}{3}\pi r f t^2$ ....(6)

Equating equations (5) and (6), there results

$$\frac{Pr}{4} = \frac{\pi r f t^2}{3} \therefore ft^2 = \frac{3P}{4\pi}.$$

or $t = \sqrt{\frac{P}{4f}}$ (approx.) ....(7)

It will be noted that in equation (7) the functions p and r are eliminated, and that the thickness of the slab is a function of the total load and fiber stress only.

Simple equations, similar to the above, may be written for variations of M_c and M_e and for materials other than plain concrete.

For a wheel load of 12,000 lbs. on a pavement foundation, using $f = 100$ lbs., equation (7) gives $t = 5.48$ ins.

For a wheel load of 20,000 lbs. and for the same fiber stress (100 lbs.), $t = 7.07$ ins.

If an earth foundation is saturated and is very soft, or if for any other reason the pressure should be reduced to the minimum, or should be made as nearly uniform as possible, a cylindrical—instead of a conical pressure—may be assumed and the following equations then will apply:

$P = \pi r^2 p$, and for a sector whose arc is unity, the load will be $\frac{pr}{2} = \frac{P}{2\pi r}$.

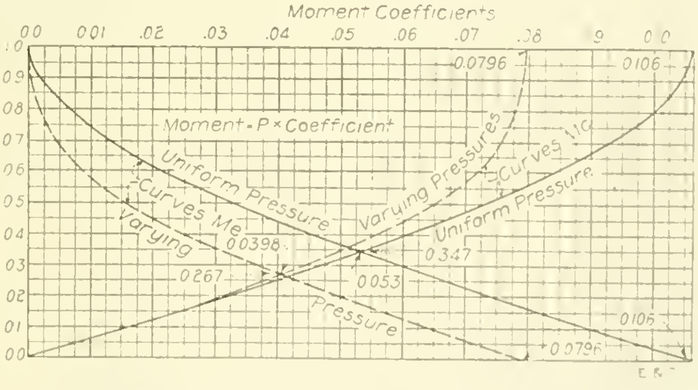


Fig. 3. Diagram for Determining Moment Coefficients for Concrete Pavement Slabs for Both Uniform and Varying (Assumed) Pressures.

The summation of pressures on a sector from the edge c to the point b (see Fig. 2) will be

$$p \cdot \frac{x}{r} \cdot \frac{r-x}{2} + p \cdot 1 \cdot \frac{r-x}{2}$$

$$= \frac{p}{2r} \cdot rx - x^2 + \frac{p}{2r}(r^2 - rx),$$

and the summation for point b to the center will be

$$p \cdot \frac{x}{r} \cdot \frac{x}{2} = \frac{px^2}{2r}.$$

Then

$$M_c = \frac{p}{2r}(rx - x^2 + r^2 - rx)x + \frac{p}{2r} \cdot x^2 \cdot \tfrac{2}{3}x$$

$$= \frac{p}{2r}(r^2 - x^2 + \tfrac{2}{3}x^2)x = \frac{p}{6r}(3r^2x - x^3).$$

And

$$M_e = \frac{p}{2r}(rx - x^2 + r^2 - rx)(r - x)$$

$$- \frac{p}{2r}(rx - x^2)\tfrac{2}{3}(r - x) - \frac{p}{2r}(r^2 - rx)\frac{r-x}{3}$$

$$= \frac{p(r-x)}{6r}(3r^2 - 3x^2 - 2rx + 2x^2 - r^2 + rx)$$

$$= \frac{p(r-x)}{6r}(2r^2 - x^2 - rx) = \frac{p}{6r}(2r^3 - 3r^2x + x^3).$$

Adding

$$M_c + M_e$$

$$= \frac{p}{6r}(3r^2x - x^3 + 2r^3 - 3r^2x + x^3) = \frac{pr^2}{3}$$

When the moments at the center and edge are equal

$$M_c = M_e = \frac{pr^2}{6}.$$

Substituting the value of $P = \pi r^2 p$ given above, we have

$$M_c = M_e = \frac{P}{6\pi} = 0.0531\,P,$$

which is the minimum value that either M_c or M_e may have; the maximum value of M_c or M_e is 0.1062 P.

For the case of a column footing and a non-continuous slab $M_e = 0$, $x = r$, and for a uniform distribution of pressure $M = 0.106\,P$.

Intermediate values may be obtained in a manner similar to that shown for a conical distribution of stress; the values of these coefficients are represented by the full lines in Fig. 3.

Assuming a case in which $M_c = M_e$, and placing their value equal to M_r for the same radius, we have

$$\Sigma M_e = \frac{pr^2}{6} \cdot 2\pi r = \tfrac{1}{3}\pi r f t^2 \therefore pr^2 = ft^2.$$

But

$$pr^2\pi = P,$$

$$\therefore P = \pi f t^2 \text{ and } t = \sqrt{\frac{P}{\pi f}}.$$

For good concrete reinforced with 0.75 per cent of steel the resistance to bending is the same as an imaginary plain concrete beam in which the safe fiber stress in tension is 600 lbs. Hence for concrete properly reinforced, distributing the pressure uniformly, when $M_c = M_e$

$$t = \sqrt{\frac{P}{600\pi}} = 0.023\sqrt{P},$$

or when either M_c or $M_e = 0$

$$t = 0.016\sqrt{P}.$$

When the conditions are inverted and the pressure is uniform, P may represent the load carried by one of a group of columns supporting a uniformly loaded floor.

If the floor is a concrete slab properly reinforced, its weight per square foot $= 12.5t + 12i$, in which $t =$ effective depth of the slab in inches, and $i =$ thickness, in inches, of concrete outside of the center of gravity of the tensile steel.

If L = load per square foot plus $12i$, then

$$P = (L + 12.5t)\,\pi r^2,$$

in which πr^2 represents the area in square feet, supported by each column.

When $M_c = M_e$,

$$t = \sqrt{\frac{(L + 12.5t)\pi r^2}{600\pi}} = \sqrt{\frac{(L + 12.5t)r}{600}},$$

which reduces to

$$t = \frac{r}{600}\left(6.25r + \sqrt{(600\,L + 39r^2)}\right) \ldots\ldots (8)$$

For the case which many engineers may prefer to use, that is for $M_c = 2M_e$,

$$t_c = 0.001923\,[6.25r^2 + r\,\sqrt{(600L + 39r^2)}\,].\,(9)$$

and

$$t_e = 0.00136\,[6.25r^2 + r\,\sqrt{(600L+39r^2)}\,]\,..\,(10)$$

Equation (9) will give the effective thickness required over the column, and equation (10) will give the effective thickness at the center of the span for the conditions assumed.

Method and Cost of Constructing a Concrete Road with Concrete Line Poles and Concrete Fence, La Salle, Ill.

A very nearly inclusive construction of concrete is represented by a stretch of road built last year near La Salle, Ill. The road is concrete and the bordering fence and transmission line poles are of reinforced concrete. The following methods and costs of this road construction are as given by Mr. A. H. Hunter, Division Engineer, Illinois Highway Commission, in a paper before the Illinois Society of Engineers and Surveyors:

The improvement of the Bottom road in La Salle township, La Salle county, Illinois, is of interest chiefly because of the completeness of design. Perhaps no road in central Illinois presents so many separate and distinct features which must be recognized when its permanent improvement is considered. It is subjected to the ravages of the Illinois River in flood time for three-fourths of its length; the remaining one-fourth is on the bluff approach to La Salle from the south and runs under the Burlington railroad viaduct up a 6 per cent grade over the Illinois-Michigan Canal and up another grade of 5.5 per cent to the bridge over the Rock Island railway tracks.

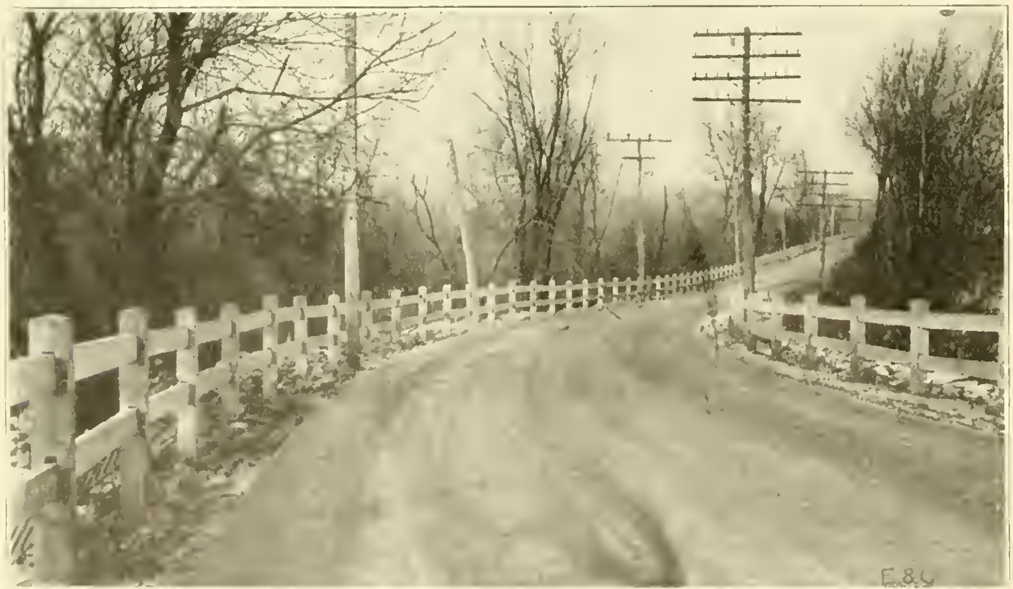


Fig. 1. Concrete Road and Fence at La Salle, Ill.

The section of the river bottom consisted of an old waterbound macadam surface not more than 22 ft. wide on top with steep slopes on either side extending to the bottom land some 10 to 12 ft. below. The other part with shale and cinder surface was restricted in width by the Burlington viaduct, and on the 6 per cent grade was only 18 ft. wide on top. Even then the toe of the side slope extended into the door-yards of adjacent property.

Traffic consisted of a large amount of heavy trucks, a fair number of pleasure vehicles and numerous foot passengers.

The improvement of the road was made possible through the efforts of the La Salle Commercial Association. Their original plan was to construct a concrete roadway with a concrete sidewalk on the east side, also guard fences and lamp posts of reinforced concrete. A request for engineering services was made to the Illinois Highway Commission and a preliminary sketch showing an outline of the road was furnished. Later a survey was made and plans drawn up for a 20 ft. concrete roadway having a thickness of 6 ins. at the side and 7 ins. in the middle, shaped to a crown of 3¾ ins. The construction of sidewalk, fences and lamp posts was not considered at that time for reasons which will be explained later.

The road was built by day labor, except hauling, which was contracted, and under the direct supervision of the Illinois Highway Commission with the writer as Resident Engineer in charge. In addition, the Commission furnished a 10-ton road roller, a concrete mixer, and an operator for each, free to the local authorities.

The concrete roadway was built upon the existing waterbound macadam surface. The old surface was loosened with a scarifier drawn by a 10-ton roller, the loose stone was then shaped to the crown of the subgrade and thoroughly rolled. An abundance of stone dust remained on top so a thorough wetting with water from the pipe line and an occasional trip of the roller prepared a sub-base which was little short of a finished waterbound macadam. This gave an excellent foundation for the concrete, and at the same time furnished a smooth surface for the shoveling of the gravel.

Drainage for the foundation was provided by longitudinal ditches 8 ins. wide and 6 ins. deep under the outer edge of the concrete. At the end of the expansion joints a lateral drain, consisting of a trench filled with gravel, led to the rip rap wall.

The aggregate consisted of clean washed gravel screened to sizes ranging from ⅜ in. to 1¼ ins. and of pure sand. The gravel consisted of granite pebbles in well graded sizes. No loam, clay or disintegrated material of any kind was present. The sand was sharp and clean but inclined to be rather fine. For the use of such excellent material acknowledgements are due Mr. P. M. Lewis, President of the American Sand & Gravel Co.

On the rolled subgrade, the loads of gravel and sand were dumped in rows. The quantity of sand and gravel per lineal foot of road was computed and the wagons measured and the loads spaced accordingly. An effort was made to have all loads the same size as this simplified matters for the man who dumped, and also relieved the engineer in charge of incessant checking up of quantities. Ten per cent excess of material was provided to allow for waste and variation in thickness. This amount proved sufficient as there was not an excess or deficiency of more than 10 cu. yds. of sand and gravel on the whole road.

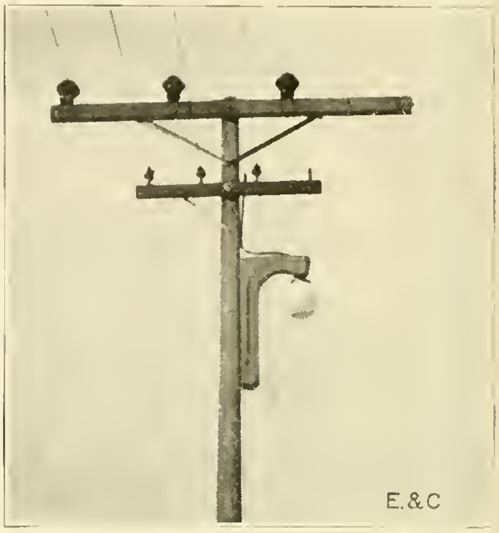


Fig. 2. Top of Reinforced Concrete Line Pole Showing Lamp Fixture, Concrete Road, at La Salle, Ill.

Cement for the south half of the road was furnished by the Marquette Cement Manufacturing Co. from their mills in Oglesby. The remaining portion of the road was constructed with cement furnished by the Chicago A-A Portland Cement Co. The quality of the cement was determined by tests of samples sent to the Testing Laboratory of the Illinois Highway Commission. All cement used comprising some 3,200 bbls. was donated by the above companies free of charge on the road.

On concrete road work a large amount of water is required, both for the mixing and for the proper curing of the finished road. It is the practice of the Illinois Highway Commission in work of this character to construct a complete system of pipe lines to which water may be furnished by city pressure or by a pressure pump operated by a gasoline engine. At La Salle, connection was made to the city system and a 1¼ in. pipe line with T's every 300 ft. was laid along the road. With 150 ft. of hose connection attached to the mixer, it was possible to secure water at all points. This plan worked well although at the extreme end of the line the pressure was not sufficient to sprinkle the finished road work while the mixer was running.

A self-moving batch mixer was used on this work. It had a low hopper in front into which sand and gravel were shoveled direct from the road bed. Two sacks of cement were used for the batch. The mixed material was deposited on the sub-grade by means of a 20 ft. rotating boom on which operated a movable bucket. This equipment made it possible to deposit the concrete on any spot of the wide base. The number of square yards laid per day of eight hours averaged 450 for the entire job. The soft concrete was struck off to crown and grade with a steel shod template cut to the crown of the roadway.

Expansion was taken care of by means of ⅜ in. asphalt joints placed 50 ft. apart at an angle of 60° with the center line of the road. The edges of these joints were protected by standard Baker plates cut to the crown of the road, with their edges flush with the surface.

The edge of the concrete was retained by wood forms consisting of 2 x 6 ins. x 16 ft. pine pieces resting on edge, held to alignment by ⅝ in. iron pins 18 ins. long. It was found difficult to butt the boards together perfectly so the more imperfect form pieces were cut in 2 ft. lengths and used for splicing. These splices were nailed to the outside of the forms with small nails. This gave good results especially on curves where it was possible to align arcs of circles almost perfectly.

The mixture used was 1:2:3½, or by volume, 1 cement, 2 of sand and 3½ of stone. A box containing 0.95 cu. ft. was constructed and for each sack of cement placed in the hopper of the batch mixer, 2 boxfuls of sand and 3½ boxfuls of stone were added. The shovels of sand and stone necessary to secure the respective amounts of material were counted and this number used for each batch.

When new shovelers were placed on the gang, the box was used to check the amount. The stone determined in this manner varied but slightly but the sand had to be changed frequently. On a dry day the amount of sand on a shovel would be smaller than usual, while on a damp, rainy day, the amount would be considerably greater. The actual amount of cement in the mix was checked every 50 ft. of road by counting the bags and computing the number of barrels necessary for the length of road.

A concrete road surface to give good and lasting results must be properly cured so that the surface coat will be tough and resist the impact of traffic. This end is usually attained, first, by a covering of canvas while the cement is setting; and second, by covering with earth to the depth of 1 in. to 1½ ins., both of which are kept moist by sprinkling. No earth was attainable as the lower edge of the fill was swampy so a straw covering was used. This proved very efficient in retining moisture. The sprinkling was continued for a period of ten days after the placing of the concrete.

The actual cost of the work was obtained by adding the daily expenditures, the cost of hauling and the market price valuation of the donated materials inclusive of freight and all incidental items. To this was added the salaries of the Resident Engineer and the two roller operators. In all 10,663 sq. yds. of surface had been improved at a cost of $13,632.16. The itemized cost was as follows:

Item—	sq. yd.
Superintendence	$0.023
Excavation, shaping road bed and trimming side roads	0.158
Hauling sand and stone	0.169
Mixing and placing concrete, setting forms and filling joints	0.133
Watchman and miscellaneous labor	0.116
Sand and stone f. o. b. siding, including demurrage	0.314
Cement, f. o. b. siding, including demurrage	0.310
Expansion joints, f. o. b. siding, including demurrage	0.028
Coal and oil for mixer and miscellaneous supplies	0.021
Forms and other lumber	0.008
Total	$1.280

The common labor used was foreign and received 35 cts. per hour and teams received 62½ cts. per hour.

The original plan included a 3 ft. concrete walk on the east side of the fill. When the road was finally located, it was found that part of the walk would come upon a new fill which was undesirable. It was decided to postpone this construction until the fill had become securely compacted. The money for this walk has been subscribed.

The plan of the lighting system was changed slightly from the first idea that was presented. A local utility company which secured the privilege of running a transmission line along the edge of the fill offered to construct their poles with a special attached arm from which a light fixture could be suspended. This scheme was adopted as is shown in the accompanying cut, Fig. 2. Conduits were placed under the concrete every 120 ft. to provide connections for additional lights which will be provided on completion of the sidewalk.

The down stream side of the fill was protected from wash by a heavy rip rap wall for in times of high water the concrete road is partly submerged. The current is very slight in most places but the lapping action of the waves eats away the unprotected edges. The flood of March, 1913, seriously damaged the upstream face. To prevent a possible recurrence of this, it is planned to rip rap the unprotected portion that remains.

It had been planned to construct the guard fence of reinforced concrete posts and two lines of 2 in. pipe rail. Later a series of experiments on the design of concrete fences were conducted by the Marquette Cement Manufacturing Co. and one of these designs was accepted. This design consisted of a 6 in. square concrete post extending above the ground about 3 ft. 6 ins., reinforced by a ⅜ in. rod placed near each corner. Provisions for panels were made by leaving slots through the center of the posts. The panels, two in number, were 2x6½ ins. concrete, reinforced by two small rods near their edges, also a 1 in. x 1 in. x ⅛ in. angle was placed in the center to prevent lateral bending.

In erecting the fence, the posts were placed 7 ft. center to center and the concrete rails slipped into the slots. This method of erection was quite simple as no cement, mortar or grout of any kind was necessary. Considerable care had to be used in aligning and setting the posts to grade or the appearance was unsightly. This fence is now in place on the west side of the fill, also a short portion on the curve approaching the shipping-port bridge, has been protected on the east side.

The construction features employed in this road are in no way different from those of many other roads, but the range of improvement shows a possibility for future development. Local conditions, together with character, amount of traffic and the type of surface required influenced the improvement selected.

Water Works

The Proposed Rock Run Storage Dam for the Additional Water Supply of Coatesville, Pa.

The present water supply of the Borough of Coatesville, Pa., is derived from a stream called Sucker Run. This supply has been inadequate for a number of years, and it is now proposed to construct works for an additional water supply to be drawn from Rock Run at a point about 2½ miles west of the city. The proposed works comprise the Rock Run storage reservoir and dam, a slow sand water filtration plant and an 18 in. pressure conduit from the dam to Coatesville. Bids for the construction of these works were received on January 15, 1914, and it is expected that construction operations will soon be under way. These improvements were designed by Mr Alexander Potter, consulting engineer, New York city, and it is from the plans, specifications and other information furnished us by Mr. Potter that the present article on the Rock Run dam has been compiled.

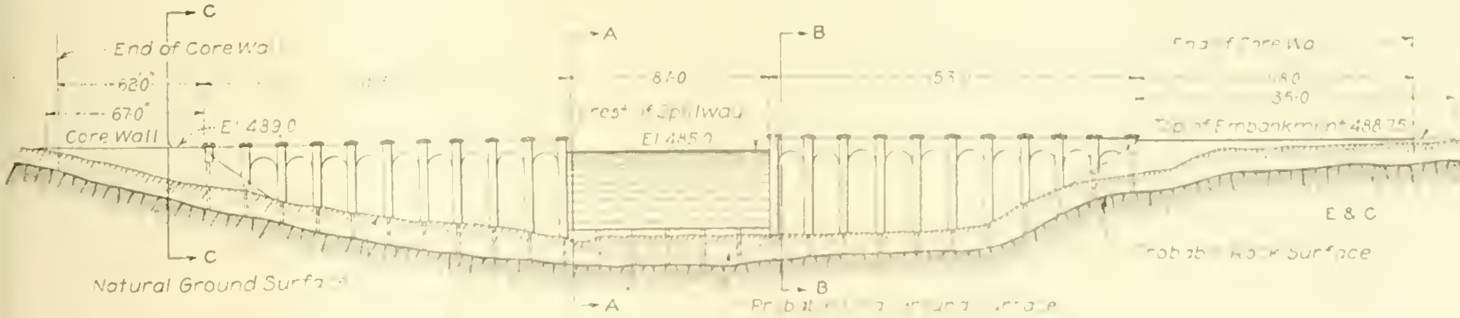


Fig. 1. Downstream Elevation of Proposed Rock Run Storage Dam for the Additional Water Supply of Coatesville, Pa.

The dam is shown in elevation in Fig. 1. It is to be built of reinforced concrete. The central portion forms a spillway 87 ft. long and has its crest placed at elevation 485. The concrete end sections are each 153 ft. long, an earth embankment is carried from the concrete end section at either end until it strikes the natural surface of the ground.

CHARACTER OF DAM SITE.

The rock surface was determined by sinking a number of open test pits down to solid rock. In August, 1913, Myron L. Fuller, a consulting geologist and expert on underground waters, of Boston, Mass., formerly in charge of underground water investigations of the U. S. Geological Survey, was engaged to report upon the proposed additional water supply. Of the character of the reservoir and dam site, Mr. Fuller reported to the Borough as follows:

Character of Reservoir Site: The rock underlying the basin consists mainly of various forms of gneiss and similar crystalline formations. These weather into a sandy soil. The proposed reservoir will cover 62.8 acres with a 35-ft. dam with crest at an elevation of 485.0. The bottom of the reservoir is relatively flat and is bordered by fairly steep slopes. The water will, therefore, be of considerable depth throughout almost the entire area of the reservoir, being shallow only along a very small area at the extreme upper end.

Character of Proposed Dam Site: The dam site selected is entirely satisfactory in character. The underlying rock is a firm, hard gneiss, affording ample stability for any structure erected upon it. It is insoluble, and no leakage can occur except through the bedding planes. Since these will run nearly parallel with the course of the dam, there is practically no opportunity for loss of water through them. The fact that the surface of the rock within the reservoir is covered by from 8 to 10 or more feet of soil will still further reduce the tendency to leakage of water. The amount escaping in this manner will, in fact, be merely seepage which may be disregarded.

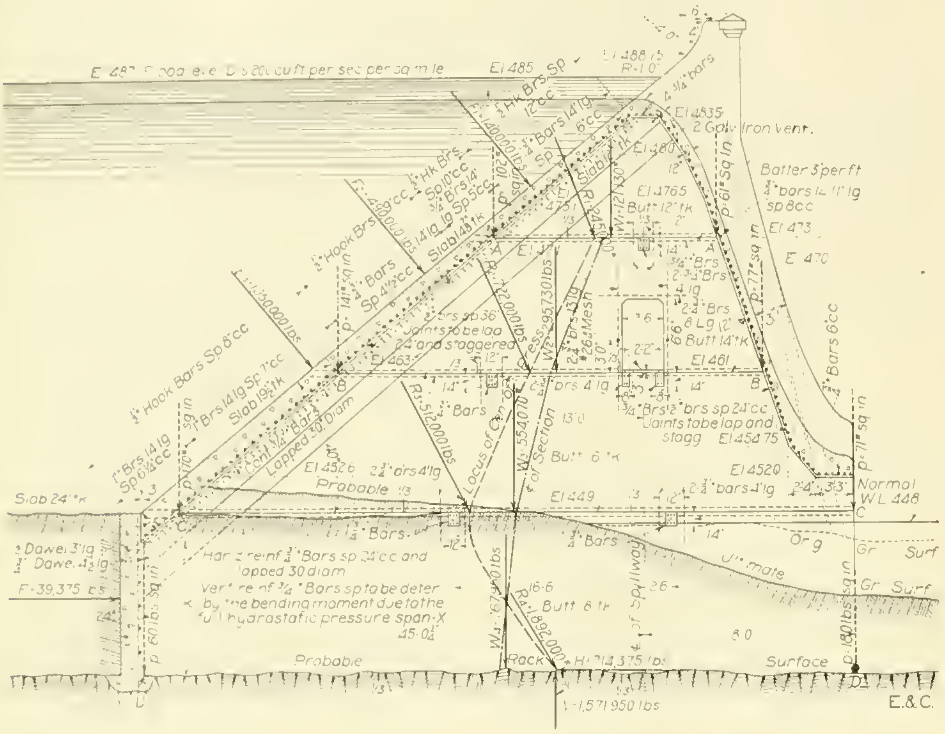


Fig. 2. Cross Section of Rock Run Dam Through Spillway, Sec. A-A, Showing Amounts and Positions of the Various Maximum Forces Which Will Act Upon It.

The rock at the dam site may be regarded as one of the most favorable of any occurring in the State.

SPILLWAY.

The spillway is of ample size to discharge the greatest flood likely to occur during the life of the structure. At a depth of 2 ft. the spillway can discharge over the crest an actual runoff of at least 200 cu. ft. per second per square mile from the tributary watershed. This, together with the storage in the reservoir, is sufficient to take care of the maximum floods. The construction of the spillway section of the dam is shown in detail in Fig. 2.

DETAILS OF DESIGN.

Both the spillway and the concrete end sections are designed to resist the full hydrostatic pressure from the maximum flood level, which has been assumed to be elevation 487.0 down to bed rock.

The deck is to be made of reinforced concrete mixed in the proportion of one part of cement, two parts of sand and four parts of broken stone. It is designed to carry in addition to its own load the hydrostatic pressure developed by the flood level at elevation 487.0

The buttresses are spaced 15 ft. centers and are corbeled out 22½ ins. on either side. The effective span length of the deck slab has been assumed to be the clear distance between the corbels plus the full depth of the slab.

Expansion joints are provided in the deck slab over each buttress to take care of shrinkage, temperature changes and settlement, without stressing the structure.

To prevent the corbels of the buttresses from splitting, due to the sliding of the deck slab on the corbel, the corbel is reinforced with ½ in. square bars hooked around ¾ in. square bars at either end of the corbel. These bars also serve the additional purpose of taking care of the tension stresses in the corbel due to cantilever action.

The cutoff wall is designed to resist the full hydrostatic pressure on the up-stream side irrespective of the earth backing on either side of it. The cutoff wall is to be built of 1:2½:5 concrete and is to be reinforced vertically with ¾ in. square bars. The wall will be held in place at the bottom by the rock footing and at the top by the deck slab.

Apron.—The apron on the down-stream side is laid out in a reverse curve. The upper portion of this curve is based on the experimental data obtained by Bazin for a crest discharge of 2 ft. The apron is not carried down the entire depth as this is considered unnecessary.

At the point where the spillway section is located the rock is at least 12 ft. below the surface. After a portion of the overlying earth is scoured out and a depth of about 30 per cent of the free fall of the water is obtained in the water cushion, no further scouring will take place.

Buttresses.—The concrete buttresses are rigidly braced longitudinally by reinforced concrete braces.

A passageway is constructed through the entire length of the reinforced concrete section and a walk with railing provided so that the interior of the dam can be inspected at all times.

The field joints in the buttresses are to be dovetailed so as to prevent all possibility of sliding.

Concrete End Sections.—The concrete end sections are similar in construction to the spillway, except that the inclined section is carried up to elevation 488.75 or 3 ft. 9 ins. above the crest.

Earth Embankments.—The construction of the end embankments, together with the core walls, is shown in Fig. 3. The earthen embankment is 18 ft. wide on the top with slopes of 1 on 2 on the up-stream side and 1 on 2½ on the down-stream side. The up-stream slope is to be paved with rip rap pavement.

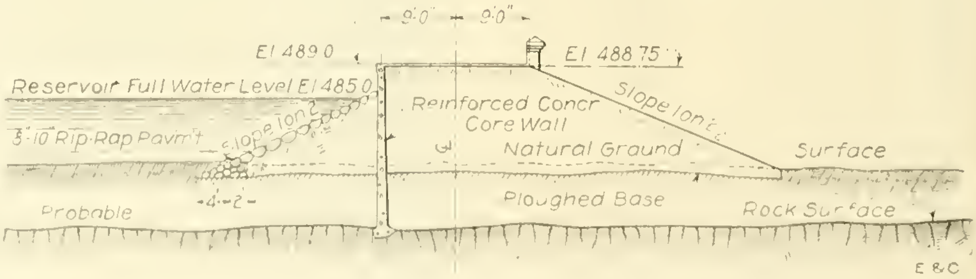


Fig. 3. Cross Section of Core Wall and Earthen Embankment of Rock Run Dam, Section C-C.

The core wall extends the entire length of the embankment and is carried up from a secure footing on the bed rock to elevation 489.0, 4 ft. above the crest level. The exposed portion of the core wall is to be provided with a reverse curve so as to prevent any water from splashing onto the earth embankment. The core wall is to be constructed of reinforced concrete mixed in the proportion of one part of cement, two and one-half parts of sand and five parts of broken stone. It is to be reinforced both longitudinally and vertically to prevent the formation of cracks such as might result from shrinkage and temperature changes or unequal settlement. The core wall is rigidly tied into the end buttress. The core wall is reinforced horizontally with ½ in. square bars spaced 12 ins. on centers above and 18 ins. on centers below the natural ground surface, respectively. The horizontal rods are in pairs, set close to each face of the wall. Just outside the horizontal bars ½ in. square vertical bars are placed 24 ins. on centers.

The end of the earth embankment facing the concrete section is held in place by a reinforced concrete wall about 18 ft. long and 12 ins. thick.

Intake and Sluices.—There are three intakes provided. The uppermost of these is 18 ins. in size and is placed at elevation 473.0. The middle one is 18 ins. in size and is placed at elevation 463.0. The lowest, an emergency intake, is 24 ins. in size and is placed at elevation 52.75. The lower or emergency intake serves primarily as a sluiceway for the dam. The outlet of this sluice pipe is supported by four wooden piles properly braced.

All the operating mechanism is placed inside the dam in the panel immediately to the northwest of the spillway section.

ARCHITECTURAL TREATMENT.

Special attention has been given to the architectural treatment of the dam. The buttresses are to be widened out 3 ft. at the back, with a batter of ⅛ in. per foot and between them at the top is to be placed a reinforced concrete arch. Between the backs of the buttresses and the concrete arch there is to be placed a sunken panel to be constructed of plaster placed on hy-rib or similar material and supported on reinforced concrete beams, somewhat similar to the construction used by Mr. Potter in constructing the Agency Hill Reservoir for the city of Muskogee, Okla. described in Engineering and Contracting for July 16, 1913.

UNIT WORKING STRESSES ASSUMED.

In designing the dam the following unit stresses were assumed:

Direct compression, 1:3:6 concrete, 200 lbs. per square inch.

Compression in bending, 1:2:4 concrete, 650 lbs. per square inch.

Tension in bending, 1:2:4 concrete, 0.0.

Shear, 1:2:4 concrete, 60 lbs. per square inch.

Steel tension in bending, 14,000 lbs. per square inch.

Ratio of modulus of elasticity of steel to that of concrete, 12.

Bond stress between steel and concrete, 85 lbs. per square inch.

Bearing power of rock for buttress footings not to exceed 15 tons per square foot.

STRESS DIAGRAM.

A section of the dam on which are shown all of the forces which act upon it, together with the resultant stresses which are set up in the buttreses of the structure is shown in Fig. 2. For the purpose of this analysis the dam has been divided into four sections marked A-A, B-B, C-C and D-D in that figure. The forces which act on the structure are indicated by solid black arrows. The forces marked F_1 to F_4 represent the total water pressure which acts on the structure above the plane in question. The forces W_1 to W_4 represent the total weight of the concrete masonry above the section, the heaviness of the concrete being taken at 150 lbs. per cubic foot. The location of the forces F and W are accurately shown.

In dotted black arrows are indicated the maximum compressive unit stresses set up in the buttress. These compressive stresses vary from a minimum of 61 lbs. per square inch to a maximum of 180 lbs. per square inch.

The forces of the type R (R_1 to R_4) show the direction, location and amount of the resultant pressures at the respective sections.

The dashed black line is the locus of the center of pressures and is so marked. This pressure curve follows very closely the center line of the buttress. Attention is called to the fact that in the design the full hydrostatic pressure has been assumed to act on the dam from the rock surface up to the maximum flood level, no account whatever being taken of the additional stability offered by the earth backing in front of and to the rear of the cutoff wall. It is due primarily to the inclination of the deck which has been taken 50° to the vertical instead of 45° as is customary, that the pressure curve follows so closely the center line of the sections. The center line of the sections is shown in the figure by a dashed and dotted line. The third points of the buttress sections are also indicated.

All of the forces shown are for a portion of the dam 15 ft. long which is carried or supported by one buttress.

The maximum vertical load on each buttress, including both the water and concrete, is 1,571,950 lbs., and the horizontal or sliding forces due to the full hydrostatic pressure from the rock surface to the maximum flood level in accordance with the above assumptions is 714,375 lbs. This requires the development of a coefficient of friction of 0.451.

COMPARISON OF COATESVILLE AND STONY RIVER DAMS.

The partial failure of the 50 ft. high Ambursen dam of the West Virginia Pulp and Paper Co., located on the Stony River at Dobbin, W. Va., which occurred on January 15 of this year, suggests the desirability of here calling attention to the distinctive points of difference between the Stony River and the Coatesville dams. From present information relative to the design, construction and partial failure of the Stony River dam and from a study of the design under consideration in this article the following are the chief points of dissimilarity between the two designs.

(1) The Coatesville dam is designed to resist the full hydrostatic pressure from the maximum flood level down to bed rock, whereas the Stony River dam was designed to resist only the hydrostatic pressure from the flood level down to the natural surface of the earth.

(2) The buttresses of the Coatesville dam will be carried down to and into bed rock, whereas the buttresses of the Stony River dam rest on a concrete footing which in turn rests on clay.

(3) The cut-off wall of the Coatesville dam will be carried down to and into solid rock, and is designed to resist the full hydrostatic pressure. In the Stony River dam the cut-off wall was only carried down to rock for the central portion of the dam, a distance of approximately 500 ft. For the remainder of the dam the cut-off wall was carried down approximately 6 ft. below the surface and not to rock which was several feet below the bottom of the cut-off wall. The Stony River cut-off wall, moreover, was built of plain concrete and therefore possessed little flexural resistance against the hydrostatic pressure.

Data on Water Purification in Illinois —Process and Plant Details.

All surface water in Illinois is subject to pollution and, at the same time, many surface streams serve as sources of water supply for cities. The purification of water for domestic use therefore becomes necessary. Many Illinois cities obtain their water supplies from wells and such supplies are, in most cases, suitable for direct domestic consumption. Some of the largest cities in the state, excepting Chicago, which derives its supply from Lake Michigan, are supplied by well water. A large number of cities are, however, forced to use surface supplies, either because a sufficient ground water supply is not available or because the ground water is not of acceptable quality. In a recent paper before the Western Society of Engineers, Messrs. Edward Bartow and Paul Hansen, Director and Engineer, respectively, of the Illinois State Water Survey, presented valuable data on water purification in Illinois with special reference to the developments in the application of the art within the State, to the processes employed and to the plants constructed for water purification purposes. Illinois supplies are so varied in character that the paper, in its application, is of far more than local interest. A large portion of the text of the paper is here given as published in the journal of the society for December, 1913.

The situation as to Illinois water supplies can be most readily understood by considering available sources of water supply throughout the State. For this purpose the State may be regarded as divided into three zones of water supply, northern, central and southern. These zones are of unequal area. In the northern zone is included the territory north of a line drawn from Quincy to the point where the Illinois-Indiana line reaches Lake Michigan. In this zone, there is generally available an abundant quantity of excellent water from deep rock wells varying in depth from about 600 ft. to 2,400 ft.

In the central zone, which lies between the line above described, and a second line extending from Danville to East St. Louis, many water supplies are obtained from deep drift wells.

The third zone, which includes the remainder of the State, contains very few wells yielding sufficient water for public water supplies. The glacial drift, where it exists at all, is too thin to produce wells with sufficient yield and the waters from rock wells are highly impregnated with mineral matter. The few exceptions to the rule are deep wells in the limestone on the southern slope of the Ozark uplift and a few deep wells in the alluvial deposits of the Ohio and Mississippi River bottoms which form the southern tip of the State.

Communities in the southern part of the State must, therefore, as a rule use surface water. Along the Mississippi River, even north of Quincy, it is difficult to secure ground water sufficiently free from mineral matter to render it acceptable for public water supplies. The Mississippi River cities, therefore, must use surface supplies, usually the Mississippi River itself. The cities along the shore of Lake Michigan use the lake water because it is most convenient and most economical.

Numerous analyses have shown that the surface water supplies of Illinois, including Lake Michigan near shore, are unsafe as sources of public water supply. The opportunities for communities to own and control entire watersheds are very few and moreover, even if water supplies could be protected in this manner, purification would still be desirable owing to the presence of large quantities of suspended matter in nearly all of the streams of Illinois. There are within the State of Illinois 68 surface water supplies, of which 33, including all the larger supplies, are now being subjected to some form of purification.

The first water purification plant in Illinois was installed at Belleville in 1886. It consisted of two Hyatt pressure filters having a capacity of 300,000 gals. per day. This plant has since been abandoned. The first plant of this type in the United States was installed in 1874. A plant having a capacity of 1,350,000 gals. per day, consisting of six 10-ft. American filters, was installed at Elgin in 1888.

Two 8-ft. American pressure filters, having a capacity of 288,000 gals. per day, were installed at Streator in 1889. In the same year two 7-ft. American pressure filters, with a capacity of 220,000 gals. per day, were installed at Rogers Park. In the same year one 10-ft. American pressure filter, having a capacity of 226,000 gals. per day, was installed at Cairo.

The first gravity filter plant was installed at Quincy in 1892. It consisted of 11 12-ft. Jewell filters, having a total capacity of 1,000,000 gals. daily.

In 1903 the first concrete gravity filter plants were installed at Danville, with a capacity of 3,000,000 gals. per day, and at Moline, with a capacity of 1,000,000 gals. per day. To 1890 seven plants had been installed; to 1905 there were 12; to 1910, 17; and since that time the increase has been rapid.

It is safe to predict that the growing recognition of the importance of having pure water supplies and the increasing demand for waters of good physical characteristics will result in the treatment of practically all surface waters within a few years.

PRESENT DISTRIBUTION OF WATER PURIFICATION WORKS.

At the end of 1911 there will be within the state 36 plants, see Table I, for the treatment of water and these will serve a population of about 490,000. This figure includes Rogers Park, but does not include a part of Chicago supplied with water sterilized by calcium hypochlorite.

Of the 36 plants, two, namely, Winnetka and Waukegan, are for sterilization with hypochlorite only, and will not be further considered. 11, including Rogers Park, are municipally owned and have a combined capacity of 13,120,000 gals. per day and serve a population of 167,100. Eighteen owned by companies have a combined capacity of 59,650,000 gals. per day and serve a population of 293,700. Three owned by the United States Government have a combined capacity of 2,800,000 gals. per day.

Considering purification plants with reference to type, there is one slow sand filter plant having a capacity of 300,000 gals. per 24 hours and serving a population of about 1,500. There are two plants utilizing sedimentation and coagulation only, which have a combined capacity of 3,700,000 gals. per day and serve a population of 17,200. There are five plants with pressure filters having a combined capacity of 6,250,000 gals. and serving a population of about 18,300. A sixth plant, at Elgin, now practically abandoned, owing to the availability of water from deep rock wells may be added to this number. This plant would raise the combined capacity to 7,600,000 gals. per 24 hours and increase the population served to 15,300. There are two plants equipped with the pressure type of filter, and so installed that they act partially as gravity filters. These plants have a combined capacity of 1,900,000 gals. per day and serve a population of 80,000. The most general type of filter is the open gravity, mechanical filter, and this type is used, or about to be used, at 22 plants having a combined capacity of 58,180,000 gals. per day and serving a population of 291,100. Two iron-removal plants designed for the removal of iron from ground waters have a combined capacity of 6,000,000 gals. per day and serve a population of about 10,000.

PURIFICATION BY SLOW SAND FILTRATION.

The first effort to purify water in Illinois by slow sand filtration was made at Rock Island in 1898. The process soon proved totally unsuited for the highly turbid and often colored water of the Mississippi River at this point. In 1911 this plant was replaced by a modern mechanical gravity filter plant. The only slow sand filter plant now in operation is at the United States Naval Training Station on the shore of Lake Michigan. The source of water supply is Lake Michigan, 1,200 ft. from shore. The plant is of approved design and utilizes sand having an effective size of 0.32 mm. and having a uniformity coefficient of 2.60. The ordinary rate of filtration is 4,600,000 gals. per day. During the first few years of operation, the Lake Michigan water was pumped directly to the surface of the filters, but the frequent high turbidity placed such a burden on the filters that during the past year a sedimentation tank, having a 12-hour period of retention, was installed. The preliminary treatment thus obtained failed to materially improve conditions and it is now proposed to assist sedimentation by means of the addition of a chemical coagulant.

PURIFICATION BY SEDIMENTATION ASSISTED BY COAGULATION.

The two plants for treating water by coagulation and sedimentation only, are located at Breese and Granite City, respectively. The plant at Breese has a nominal capacity of about 200,000 gals. per 24 hours and a sedimentation period of about 40 hours. Lime and iron are used as coagulants. This plant has been in operation something less than a year, and up to date has proved very inefficient. Part of the inefficiency may be explained by certain defects in design, but even with these defects corrected and with intelligent and painstaking operation, there will still be difficulty in securing as good results as have been obtained at the St. Louis water purification plant, after which this plant was modeled.

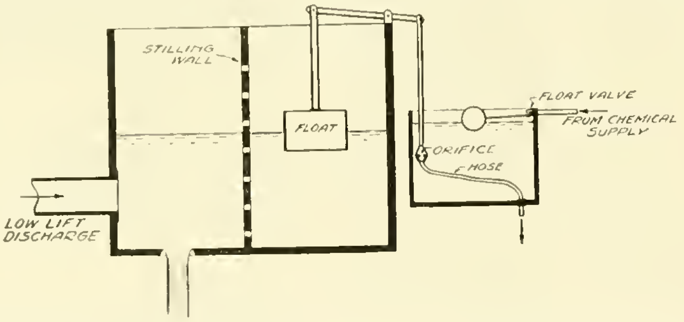


Fig. 1. Simple Device in Use at Mt. Carmel, Ill., Water Filtration Plant for Feeding Chemical Solution at a Rate Proportional to Rate of Raw Water Flow from Low Lift Pump.

The other installation is at Granite City, where a most interesting treatment is adapted to the peculiarities of local conditions. The water supply is drawn partly from the Mississippi River, partly from a slough formed in an old channel of the Mississippi, and partly from tubular wells yielding water containing large quantities of iron. The Mississippi River or the slough water is mixed with the ground water in proportions found by experience to be suitable, and to the mixture is added a quantity of lime. The lime causes a rapid precipitation of ferric hydrate and also partially softens the water. The total sedimentation period is 55 hours. The plant will soon be abandoned and Granite City will receive its water from the East St. Louis water works.

MECHANICAL FILTRATION.

Filter Plants of the Pressure Type.—The results obtained by means of pressure filters are more or less unsatisfactory. At McLeansboro, population 1,800, there is a small pressure filter with a nominal capacity of 240,000 gals. per day, provided with neither coagulating basins nor clear-water storage. The water as it comes from the filters is generally turbid and the bacterial removal is very low. The plant could be improved by modifications in the coagulant preparation and feed devices, but it is not probable that it will give entire success.

There has been greater success at Kenilworth, where there are pressure filters having a nominal capacity greatly in excess of the consumption. At Lake Forest a safe water is obtained with pressure filters assisted by sterilization with hypochlorite, but at times the water in the mains has been turbid. The good results at Lake Forest must be in large part ascribed to the maintenance of systematic analytical control.

The new plant at Fort Sheridan has a capacity of 1,500,000 gals., but it is rarely called upon to supply more than 500,000 gals. It has not been under observation long enough to determine its success or non-success. In the absence of coagulating basins, much difficulty has been encountered in securing good coagulation of the water, especially in cold weather, and it is probable that some of the alum passes through into the filtered water. The bacterial results may be maintained reasonably low by the use of calcium hypochlorite, for which the plant is equipped. With the present small consumption, the plant, if properly operated, should be capable of delivering a reasonably clear and safe water to the consumers.

Modified Pressure Filters.—Filters of the pressure type, but operated as gravity filters, are used at East St. Louis and Streator. They are enclosed in steel shells just as are ordinary pressure filters, but otherwise they operate in the manner of gravity filters; that is to say, the water flows into them under a head of a few feet from the coagulation and sedimentation basins and the effluent passes into a clear-water reservoir. Thus the filters are not at all subject to the influence of the pumps. When such filters are properly designed and properly controlled, they give good results, but they have the serious disadvantage of relative inaccessibility. Both of these plants are under careful management, and available data indicate that they produce regularly acceptable results.

Mechanical Gravity Filters.—The open gravity type of filter has come to be recognized as the standard installation, and, as will be observed in Table I, the great majority of plants are of this type. Within the State of Illinois may be found plants representative of almost every stage in the development of gravity filters. The old plants at Quincy and Decatur, both now being replaced by new and larger plants of most modern construction, are examples of early practice. At Quincy in the nineties a series of important experiments were conducted which greatly increased our knowledge with reference to the use of lime and iron as coagulants. From these we pass through later developments at Cairo, Pontiac, Alton, Danville, Moline, and finally reach the most up-to-date installations now in use at Rock Island and under construction at Quincy, Decatur and Evanston.

A group of the older plants, including the Cairo plant and the old plants at Quincy and Decatur, give poor results, primarily because they are now worked beyond their normal capacity. At the present time, however, these old plants are generally producing a safe water as a result of the addition of calcium hypochlorite as a sterilizing agent.

There are plants at Pontiac, Warsaw, and Lawrenceville of old design, but of comparatively recent installation. The Pontiac and Lawrenceville plants are faulty in design and operation, but hypochlorite is now maintaining the water safe in each instance.

COMPONENT PARTS OF MECHANICAL FILTER PLANTS.

Mechanical filtration in Illinois can be most advantageously discussed in a brief space by discussing the several component parts in the light of Illinois practice rather than by describing plant by plant.

Coagulant Preparation and Feed Devices.—The chemical preparation and feed devices constitute a most important part of a mechanical filtration plant, though perhaps the least costly part. It is essential that the chemicals be added in the proper quantities and uniformly. But little real advance has been made in the design of the chemical preparation and feed devices. An early type, as exemplified in the Egyptian water wheel arrangement at Decatur, still gives very satisfactory results as compared with the more modern designs. This device consists of a dipper wheel made of bent piping, revolving partially submerged in a tank full of solution, the surface of which is maintained at a constant level. The wheel is actuated by a propeller placed in the raw water inlet pipe, so that it revolves and dips up solution at a rate approximately proportional to the volume of water entering the plant. The solution, which is made of a suitable strength to secure the requisite application to the raw water, flows out through the hollow hub of the dipper wheel and is conveyed by suitable piping to the desired point of application.

A very common and generally satisfactory device for maintaining a uniform flow of coagulant is the so-called constant head orifice box. In this device a constant head of solution is maintained over a free discharging orifice of sufficient size to deliver the requisite quantity of solution. In the more modern types, the orifice is made adjustable so that the application of chemical may be varied without changing the strength of solution. A somewhat ingenious modification of the principle involved in this device is found at Alton. This apparatus is more compact than the ordinary constant head orifice box and is arranged for ready flushing out with clear water when desirable.

At Mt. Carmel is a simple form of apparatus, Fig. 1, which applies the chemical at a rate directly proportional to the volume of water entering the plant. It performs the same function as does the Egyptian water wheel above described, but somewhat more accurately. The water entering the plant is received in a small chamber with an orifice in the bottom. Within the chamber is a float connected to one end of a lever arm, the other end of which is connected to an orifice attached to the end of a flexible pipe placed within a constant level solution tank. The apparatus is so adjusted that the head of water over the orifice in the inlet chamber to the plant is a fixed multiple of the head of solution over the orifice in the solution box.

At some of the plants in the State, notably at Cairo and at the new Evanston plant, solution pumps will be used to discharge the chemical into the raw water at the proper rate. The chemical pumps at Ft. Sheridan and Waukegan are actuated by the pulsations of the service pumps by means of water connections. Their rate of operation is, therefore, proportional to the rate of operation of the service pumps.

Plain Sedimentation Basin.—Plain sedimentation is an unusual preliminary to chemical coagulation and sedimentation and filtration, but appears to have considerable value in the treatment of water carrying heavy turbidities in that it economizes on coagulant and produces a more uniform water to handle. Preliminary sedimentation is systematically used at East St. Louis, where the retention period is approximately ten hours. At Rock Island it is also used, the retention period being about twelve hours; the arrangement is such, however, that coagulant may be added at the entrance of this basin when desired. At Mt. Vernon and Pana a prolonged period of sedimentation is had in impounding reservoirs, but these cannot be regarded as parts of the filtration plant.

Reaction Chambers.—Within the last few years the reaction chamber has come to be recognized as an important part of a mechanical filter plant. Its function is to maintain the water in gentle agitation after the coagulant is added until the reaction is complete. Five plants in Illinois include this feature, namely, those at Evanston, Decatur (new plant), Quincy (new plant), Charleston, and at Mt. Vernon. All these plants are now under construction or were completed during 1913. Practice varies with respect to the retention period from 8 minutes at Charleston to 19 minutes at Evanston. The reaction chamber is generally a conveniently shaped compartment adjacent to the coagulation basins and highly baffled to produce a velocity of between ½ ft. and 1½ ft per second. The baffles are generally of wood and so arranged that they may be moved from time to time to meet the requirements of varying quantities of water treated by the plant.

Coagulation and Sedimentation Basins.—The general tendency in filtration practice in recent years in this State, as well as in others, has been to increase the retention period in the coagulation and sedimentation basins. These basins must, of course, be designed with special reference to the water to be handled and may vary within wide limits because of this consideration. On the other hand, the advent of the reaction chamber has probably checked greater increase in size because this adjunct now performs more efficiently and in less space what sedimentation basins were formerly counted upon to do.

Notwithstanding the fact that coagulating and sedimentation basins have been regarded as more and more of a necessity, and are counted upon to do more and more of the work of purification, thereby constituting filtration a final though very important finishing process, yet some of the latest designs include no coagulation and sedimentation; for instance, those at Pana, Ft. Sheridan, and McLeansboro. Aside from these exceptions, coagulating basins are used, and the periods of retention vary from 1½ hours at Danville to 18 hours at East St. Louis. The new plant at Evanston will have basins with a retention period of 1.7 hours. Here the turbidity will be only occasionally heavy and never difficult to handle. The new plant at Decatur will have basins of four hours' retention to handle the moderately troublesome water of the Sangamon. At Quincy 4.6 hours will be used, combined with much elasticity in operating arrangements, to meet the requirements of the very troublesome water of the Mississippi River.

Coagulation and sedimentation basins are generally provided in duplicate, and sometimes these basins are of different sizes to give greater range in the retention periods available. This latter arrangement obtains in the new plant at Quincy. In the larger and more modern plants, the basins are arranged so they can be operated in series or in parallel, with or without the reaction chamber, and sometimes still other combinations are provided. In the smaller plants, however, the tendency in the design of sedimentation and coagulation basins is toward simplicity. With the advent of the reaction chamber even the duplicate basin is omitted, as this will afford all necessary preliminary treatment during clear-water periods.

Filters.—The filters proper vary greatly in appearance, ranging from the old Jewell high wooden tanks 12 ft. in diameter and 12 ft. high installed in 1891 at Quincy, to the 36 ft by 23½ ft concrete units of the new Evanston plant. With the exception of the old Warren filters at Decatur installed in 1893, there is very little essential difference in the interior arrangements between the old and the new filters. The difference lies almost wholly in progressive improvements in details of design. All have in their bottoms a grid or system of conduits with perforated brass plates or strainer heads for collecting filtered water and distributing wash water. All have a sand bed of 2½ to 3 ft in thickness. All have the usual connections for admitting raw water at the top, for drawing off filtered water at the bottom, for filtering to waste, for admitting clear wash water at the bottom and drawing off soiled wash water at the top. All operate with a range of filtering head up to 10 ft. The greatest differences lie in the method of agitating the sand. The older types have the cumbersome revolving rakes for agitating the sand bed, while the later designs are agitated by air blown in at the bottom, or rely entirely on the wash water when applied at a rapid rate. The old-fashioned rakes are not, however, regarded as obsolete and only during the past year a small plant at Warsaw was equipped with this agitating device.

The old Warren plant at Decatur represents a variation from the general type, in that the filters have double bottoms instead of grids for conduits, and there is a continuous perforated sheet of brass over the whole bottom of each filter instead of the small perforated strainer heads or plates. There is also a very thin bed of very coarse sand or crushed quartz not over 22 ins. thick, and, to offset the slight frictional resistance thereof, the filters are not permitted to operate under a head greater than 2 ft. There is also a difference in the relation of the several filters to each other, more particularly in the mode of control and washing. But as this type of filter is practically discarded and only of interest from an historical standpoint, further description will be omitted.

CHIEF DIFFERENCES IN DESIGN BETWEEN OLD AND NEW TYPES OF FILTERS.

The most pronounced differences in design, aside from those already mentioned, between the old and new types of filters are:

First.—The change from the circular to the rectangular form which adds nothing to the efficiency, but greatly improves the general layout and is economical of space and material.

Second.—The increase in size of units which likewise adds nothing to efficiency, but makes for economy of construction and facility of operation.

Third.—The introduction of a central gutter, an outgrowth of the increase in size of units which virtually divides a filter into two units, though still retaining a single set of inlet and outlet connections. This adds noth

TABLE I. LIST OF WATER PURIFICATION PLANTS IN ILLINOIS, 1913.

City.	Population.	Source of Supply.	Ownership.	Reaction Chambers. — Minutes.	Coag. Basins. — Hours.	Filters. — M. G. P. D.	Clear well. Hours.	Type of plant.	Chemicals used.	Cost of plant per M.G.D.
Rock Island Arsenal	1,000	Mississippi river	U. S.	...	1.5	1.0	1.5	Rapid sand	A-H	.....
Kenilworth	1,000	Lake Michigan	P	...	...	.5	...	Pressure filters	A	....
Naval School	1,200	Lake Michigan	U. S.	...	12.0	.3	20.0	Slow sand	A-H	......
Ft. Sheridan	1,500	Lake Michigan	U. S.	...	...	1.5	...	Pressure filters	A-H	.....
Hamilton	1,700	Mississippi river	M	.3	3.0	1.5	2.5	Rapid sand	A	.....
McLeansboro	1,800	Reservoir	M	...	...	.24	...	Pressure filters	A	....
Breese	2,200	Kaskaskia river	M	...	40.0	200,000 Pumpage	24.0	Coagulation and settling	L-I	......
Warsaw	2,200	Mississippi river	M	...	4.0	.18	2.0	Rapid sand	A-H	.....
Winnetka	3,500	Lake Michigan	M	...	...	...	...	Sterilization	H	...
Carlinville	3,700	Macoupin creek	P	...	5.0	.5	5.0	Rapid sand	A	..
Lake Forest	4,000	Lake Michigan	P	...	...	1.0	...	Pressure filters	A-H	.....
Lawrenceville	4,000	Embarrass river	P	...	3.0	.65	5.0	Rapid sand	A-H	...
Charleston	6,000	Embarrass river	M	8.0	2.8	1.0	3.0	Rapid sand	A-H	.....
Macomb	6,000	Crooked creek	M	...	4.2	1.0	3.6	Rapid sand	A	.....
Pana	6,500	Reservoir	M	...	...	1.0	1.3	Rapid sand	A-H	....
Pontiac	7,000	Vermilion	P	...	1.2	1.5	1.8	Rapid sand	L-I-H	....
Mt. Carmel	8,000	Wabash	P	...	2.7	1.5	.75	Rapid sand	A-H	$ 6,700
Murphysboro	8,000	Big Muddy river	P	...	8.4	2.0	4.0	Rapid sand	L-I	....
Mt. Vernon	9,000	Reservoir	P	13.0	4.0	1.5	1.2	Rapid sand	A-H	....
Rogers Park	10,000	Lake Michigan	M	...	...	3.0	...	Pressure filters	A	....
Kankakee	14,000	Kankakee river	P	...	5.0	4.0	2.0	Rapid sand	A-H	.....
Cairo	15,000	Ohio river	P	...	3.8	3.0	.5	Rapid sand	L-A-H	.
Granite City	15,000	Wells, slough and Mississippi river	P	...	55.0	3.5 M. G. D. Pumpage	.	Coagulation settling and softening	L-H	
Streator	15,000	Vermilion river	P	...	...	2.5	...	Rapid sand	A-H	
Alton	18,000	Mississippi river	P	...	5.0	5.4	3.0	Rapid sand	L-I-H	
Freeport	18,000	Wells	P	...	1.4	2.0	2.3	Sedimentation and filters—Iron removal	L	
Waukegan	18,000	Lake Michigan	M	...	...	..	...	Sterilization	H	$ 300
Urbana and Champaign	22,000	Wells	P	...	3.0	2.0	9.0	Aeration and rapid sand—Iron removal		
Evanston	26,000	Lake Michigan	M	19.0	1.7	12.0	4.0	Rapid sand	A-H	.....
Rock Island	26,000	Mississippi river	M	...	12.0	6.0	24.0	Rapid sand	A-H	$10,800
Elgin	27,000	Wells and Fox river	M	...	...	2.3	...	Pressure filters	A	
Moline	27,000	Mississippi river	M	...	2.5	6.0	3.0	Rapid sand	A-H	
Danville	30,000	Vermilion river	P	...	1.5	4.5	2.4	Rapid sand	L-I-H	
Decatur	35,000	Sangamon river	M	16.0	4.0	9.0	1.0	Rapid sand	A-H	$1[illegible],400
Quincy	37,000	Mississippi river	P	16.5	4.6	6.0	.7	Rapid sand	A-L-I-H	$11,100
East St. Louis	65,000	Mississippi river	P	.	18.0	16.5	3.4	Rapid sand	A-L-I-H	

Legend: A—Alum. L—Lime. I—Iron. H—Hypo. M—Municipal. P—Private. U. S.—U. S. Government.

ing to the efficiency and is a mere convenience of arrangement.

Fourth—The introduction of concrete collection and distribution systems, which cheapens cost of construction and permits of readily using larger conduits, thus reducing frictional resistance.

Fifth—The more careful design of troughs for carrying off soiled wash water so that the same may be carried off evenly and rapidly from all parts of the filter. This improvement was partly imposed by increased size of units, but also adds to the effectiveness of washing.

Filter Control.—In the older plants, the filters are always controlled by hand-operated valves which renders the washing of filters a time-consuming and awkward operation. In the larger modern plants hydraulically-operated valves are the rule. This greatly simplifies the labor of cleaning a filter, permits of a simpler and better arrangement of the pipe gallery in front of the filters, and makes possible a neat-appearing operating floor. The main objection to hydraulically-operated valves is the annoyance of "sticking." This can, however, be largely overcome by using parallel-seated self-adjusting disk valves and by systematically cleaning the hydraulic cylinders at proper intervals.

The necessity for controlling the rate of filtration automatically is becoming more and more recognized. Even early filters had means of control, but until recently many filter operators held that an occasional partial turn of the effluent valve gave sufficiently close control. This is in large measure true, but this method places too great reliance on the watchfulness and reliability of the filter attendant. Most of the plants in the State are without automatic rate-controllers. There are three general types of filter rate-controllers in use, namely, the float type, the disk type, and the meter type. The float type in principle is simply a device for automatically maintaining a fixed head over an orifice of definite size or over a weir, generally the former. This type of controller is exemplified by those at Cairo, Pana, Moline, and Danville. The disk type maintains a constant rate of flow by producing a constant head on a submerged orifice by means of properly weighted disk and balanced valve fastened to the same shaft. The dropping of the disk causes the valve to open wider and the raising thereof causes it to close. These controllers have a tendency to stick, but when carefully made and provided with properly designed valve ports, they give good results. Controllers of this type are installed at Rock Island, Macomb, Alton, Mt. Vernon and Charleston. The meter type of controller depends for its operation on maintaining a constant difference of head between the throat and entrance to a Venturi tube. Any variation is transmitted to a diaphragm which actuates a pilot valve and which in turn operates a gate valve. The variations in head may also be transmitted by electrical means to the gate valve. Controllers of this type are in use at Ft. Sheridan and are about to be used in connection with the two new plants at Quincy and Decatur.

Loss of head gages are devices for observing the loss of head suffered by the water in passing through a filter. Therefore, they indicate the condition of the filter sand beds and give the attendant a means whereby he can ascertain when a filter requires washing The loss of head gages are of two types, namely, the indicating and the recording. The indicating are suitable for ordinary purposes, but there is a great advantage in having a record of the filter runs. Loss of head gages are not in as general use as they should be and in some plants they are permitted to become dismantled. Those plants having loss of head gages of the indicating type in use are Moline, Kankakee, Mt. Vernon, Macomb, Charleston, Warsaw, Hamilton and Ft. Sheridan, and Rock Island Arsenal. Rock Island has recording loss of head gages.

Clear Water Storage.—The provision made for clear-water storage varies greatly. In some plants it is scarcely more than a suction pit, while in others the storage is as much as 24 hours. The size of clear-water basins will depend largely on the storage available in distribution and equalizing reservoirs connected with the distribution system. For example, at Quincy the retention period is only 0.7 hour in the clear-water basin, but there is a reservoir holding about 14 days' supply on the distribution system. At Rock Island the clear-water basin, together with the filter plant, is at a high elevation and is directly connected with the distribution system. The storage period is 24 hours.

Generally speaking, there is a tendency to make the clear-water basin too small, when it is considered that the filters must operate at a uniform rate no matter what the daily and hourly rate of consumption may be.

Accessories.—Filter plants have a number of important accessories, the design and arrangement of which vary considerably in practice. As a rule, water from the source of supply must be raised to the filter plant ordinarily through a low lift. For this purpose electrically or steam-turbine driven centrifugal pumps are nearly always used because of their special adaptability to such service. The quantity of water pumped and the head pumped against are constant or nearly so and the total lift is not great. The water often contains grit and is often hard on piston and plunger pumps.

Wash water is supplied to filters in several ways, namely, (1) by drawing from the distribution system, (2) by installing low-lift centrifugal pumps taking suction from the clear well, each with a capacity sufficient to furnish wash water for one filter, and (3) by having an elevated storage tank supplied by small pumps starting and stopping automatically or by having a connection with the distribution system. In modern practice the storage tank is considered the most economical and desirable and is the method adopted for all of the new and larger plants now under construction. In all these cases small automatically-controlled pumps will be used, but there will be in the case of the Quincy plant an emergency connection to the distribution system.

In all of the plants in this State where air agitation obtains, blowers are used, but in modern practice elsewhere compressors and air storage tanks are in favor.

Storage of chemicals is an important factor and often adequate provision is not made therefor. The storage space desirable will vary with local conditions, character of water to be treated, shipping facilities, etc., but even in small plants there should be sufficient storage available to permit of buying in carload lots. In the larger plants, large storage may be advisable in order to take advantage of market conditions.

Every filter plant should have some provision for office space, laboratory, and toilet and locker rooms. Too often these matters are neglected in the smaller plants and where proper conveniences are lacking the operation of the plant is almost certain to suffer.

A word may be said with reference to the appearance of superstructures. Progress in this connection is being made, for it is unusual nowadays to see wooden or cheap brick barn-like structures built over large and important plants. But the superstructures for the smaller plants are still very crude in design. The additional expense for rendering these structures attractive is small compared with the cost of the whole plant, and it is eminently fitting that water works buildings in general and filter buildings in particular should present a neat, clean and inviting appearance.

GENERAL ARRANGEMENT OF FILTER PLANTS.

The arrangement of mechanical filter plants varies greatly, depending on size of plant, local topography, and other local requirements and depending on the ideas of the designing engineer. A plan that meets with general favor is what may be called the head house plan in which the several parts of the plant are grouped about a more or less centrally located head house. The raw water is received at the head house, treated with chemicals, diverted to the reaction chambers and coagulating basins, then returned to the head house and diverted to the filters. The head-house superstructure also houses chemical preparation and feed devices, machinery, laboratories, toilet rooms, locker rooms and offices. In some cases the head house is large enough for storage of chemicals also.

Frequently filter plants are not designed with a proper consideration for future extension and this in one or two instances has caused much awkwardness in making additions.

Laboratory Control.—Laboratory control is feasible and necessary in even the smallest plants to secure best results. Under actually existing conditions, laboratory control is especially necessary in the small plant because such plants are as a rule less perfect in design and construction than the larger ones. The tests required are very simply carried out and can, if necessary, be entrusted to a man of ordinary intelligence and industry, even though he has no scientific training. The determinations that should be made are turbidity, color, alkalinity, the number of bacteria and gas formers in the raw and filtered water. Solutions should also be tested, especially hypochlorite solutions. Should there be a municipal or industrial laboratory near by, it may perhaps be better in the case of small plants to have the bacterial work done there, but there is no reason why the simple physical and chemical tests above enumerated should not be made in even the smallest plants.

In conclusion, it is interesting to note that wherever reliable records can be obtained they show a marked reduction in typhoid fever even where the efficiency of the plant is not as great as it should be. At Decatur there have been no accurately kept typhoid fever records, but the city has a reputation of being quite free from the disease. Much of this must be attributed to the old Warren filter plant, though its average efficiency, based on bacterial removal during late years, was only 90 per cent. At Moline and Rock Island the typhoid death rate has been low since the installation of filters. Shortly before placing the new plant at Rock Island in regular operation, there was a serious epidemic of typhoid, showing the great danger that lurks in the use of the untreated river water.

It is unfortunate that there is not in Illinois a better system for the registration of vital statistics, since these form the best means whereby we may judge the efficiency of water purification works as well as estimate the need of such works.

Notwithstanding this lack of good vital statistics, the many advantages of having a clear, colorless water make it safe to predict that within a few years we will see the purification of all muddy and colored surface water supplies.

Chicago Cement Show.—There will be held in Chicago during the Cement Show, Feb. 12-21, conventions of five organizations: viz.: American Concrete Institute; National Builders' Supply Association; Interstate Cement Tile Manufacturers' Association; Illinois Lumber and Builders' Supply Dealers' Association, and the Illinois Association of Municipal Contractors. The National Conference on Concrete Road Building will also be held during this period in conjunction with the show.

Scenic Road to Mount Revelstoke.—A scenic road is being built from Revelstoke, B. C., up Mount Revelstoke to Victoria Park on its summit. Commencing on the western boundary of the city, at an altitude of 1,450 ft., with the aid of five switchbacks, the road reaches the summit, 12.8 miles distant, at an altitude of 6,150 ft. Upon completion of the road the Canadian Pacific Railway contemplates the establishment of an automobile service connecting its hotel at Revelstoke with the chalet at the summit, which is to be constructed by the company. The average grade of the road is 6.6 per cent and the maximum grade 7.6 per cent. The park on the summit consists of a plateau of some 70,000 acres.

Bridges

Design and Construction of the Bush and Gunpowder River Bridges Consisting of a Series of Reinforced Concrete Slab Spans.

(Staff Article.)

Owing to the extremely soft character of the sub-soil and to the depth of a stable foundation material, an unusual design was adopted for the bridges over the Bush and Gunpowder Rivers, which recently have been completed. These bridges are located on the Philadelphia, Baltimore and Washington division of the Pennsylvania Railroad between Baltimore and Washington. They are double-track structures, and replace wooden trestles which have become too light for the existing traffic and which have required heavy maintenance charges. The bridge over Bush River has a total length of 2,751 ft. 10 ins., and that over the Gunpowder River a total length of 4,961 ft. 6 ins. The total width of each of these bridges is 30 ft. 8 ins. The type of construction adopted for each span is the same—short-span reinforced concrete slabs supported on concrete piers. Each bridge has a bascule span, with a clear opening of 35 ft. The construction of these bridges required about 65,200 cu. yds. of concrete, 13,000 piles, and about 4,200 tons of reinforcing steel. The contract price of both structures was about $1,250,000.

LOCATION OF BRIDGES AND CHARACTER OF SUB-SOIL.

The Bush and Gunpowder Rivers, which are spanned by these bridges, are inlets of the Chesapeake Bay. These rivers are separated by a long irregular-shaped body of land known as Gunpowder Neck. The railroad line crosses each river about seven miles above its mouth. The maximum depth of water at low tide, at the location of the Gunpowder River bridge, is about 5 ft., while that at the Bush River is about 10 ft. The range of tide is about 1½ ft., and there is very little current in the rivers. At times during a very high wind from the north much of the water is driven into the bay, and parts of the river bottoms are exposed. The alignment of both bridges is tangent, and the tracks are level.

As the character of the sub-soil and the type of construction adopted for both bridges are the same, most of the following description will apply directly to the Gunpowder River bridge. At the site of this bridge 21 borings were taken to determine the character of the sub-soil. Some of these borings were taken on the center line of the new bridge, some a short distance north of it, and some were taken a short distance south of the old trestle. At the west end the distance between the center lines of the old and new structures is about 60 ft., and at the east end it is about 102 ft. Although a maximum depth of over 100 ft. was reached by these borings no stable foundation material was found. The soil directly below the surface of the water consists of soft mud, this being underlaid with strata of clay, sand and gravel.

Figure 1 shows the results of borings taken at three different locations along the bridge. The vertical scale for the drawings is ten times the horizontal scale. The piers, which are located 26 ft. apart except at the lift span, are numbered from the west end consecutively to the east end. Figure 1 (a) shows a plan and elevation of borings taken at the west end of the Gunpowder River bridge; Fig. 1 (b) shows a boring taken about 115 ft. south of the center line of the new bridge and opposite the lift span (between piers Nos. 144 and 115); and Fig. 1 (c) shows several borings taken near the east end of the bridge. The remainder of the 21 borings were taken between those shown in Fig. 1. The depth of the water at several points along the bridge can be noted by referring to the drawing.

FACTORS GOVERNING THE DESIGN.

Although there is no dredged channel in either river and the depth of water will permit only the passage of small boats, the government required a movable span of 35 ft. in the clear for each bridge. The position of the bascule span for the Gunpowder River bridge is shown in Fig. 1 (b). This was the only requirement other than the desirability of securing a permanent and economical bridge. A study of the results of the borings showed that a bridge with long spans was unpracticable on account of the excessive depth of a good foundation material. It was therefore decided to design a structure with relatively short spans, which would solve the problem of distributing the load over the supporting soil.

As the bridge was to be of permanent construction it was decided at the start to use a solid floor. Three types of bridge were considered: (a) A series of short-span concrete arches; (b) a series of deck plate girders; and (c) reinforced concrete slabs on concrete piers. The arch type was disregarded, both on account of high first cost and probability of considerable settlement of the piers. After making an estimate of cost of the remaining two types and capitalizing the extra cost of the plate girder type (taking into account maintenance costs), it was decided that the reinforced concrete slab type offered the best solution for these bridges. As the bridge was a double track structure it was necessary to use comparatively short spans to avoid large, spread foundations. By using a pile foundation it was determined that a span length of 26 ft. would permit a foundation to be designed whose width would not greatly exceed that required for the roadway construction. Although some of the piles were only about 30 ft. long, almost 70 per cent of them had a length of between 60 and 70 ft. They were figured to carry a load of about 14 tons each.

The slabs are not made continuous over the supports, but are designed to act as simple spans, to avoid any danger due to unequal settlement of the piers. The span length of 26 ft., center to center of piers, is used for all spans except at and adjacent to the bascule span. At this span the piers are spaced 42 ft. 6 ins. apart to provide a clear channel of 35 ft. Adjacent to the bascule span there is a 30-ft. plate girder span to provide a support for the counterweight and operating machinery, while next to the lift and girder spans the piers are spaced 26 ft. 8 ins. apart. In the Gunpowder River bridge there are 186 typical 26 ft. spans, and in the Bush River bridge there are 101 typical 26-ft spans, in addition to the four spans for each bridge above mentioned.

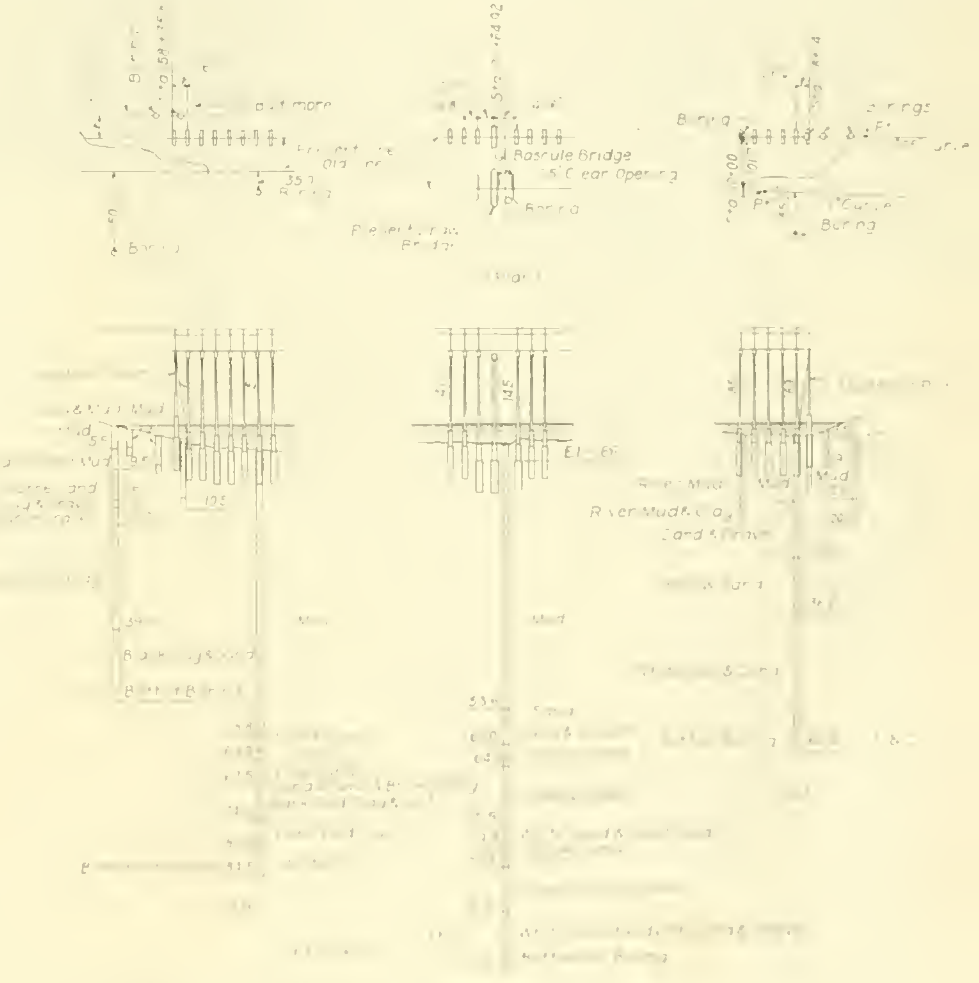


Fig. 1. Results of Borings at Several Locations Along the Gunpowder River Bridge.

REINFORCED CONCRETE FLOOR SLABS

Loads and Stresses.—The reinforced concrete simple spans have a 1-in. space between them to insure their acting independently. The live load used is that given in the 1912 specification of the Pennsylvania Railroad. The total weight of engine and tender is 241 tons, with axle loadings of 35 tons. The impact allowance for the 26-ft. spans is 48 per cent of the live load. The track construction consists of 100-lb. rails, 7x10-in. x 8-ft. 6-in. ties, and broken stone ballast, the distance from base of rail to top of slab being 14 ins. The tracks are spaced 13 ft. on centers, and it was assumed that each track distributed its load over a width of 13 ft. Under the above assumption the dead, live and impact loads gave a maximum moment of 1,656,000 in.-lbs. per foot of width and a maximum shear of 21,850 lbs. per foot of width.

The allowable tensile stress in the reinforcing steel was 12,000 lbs. per square inch, and the allowable compressive stress in the concrete was 600 lbs. per square inch.

Principles of Design.—In designing the floor slabs the ratio of the modulus of steel to that of concrete was taken as 15, and the straight line formula was used for the distribution of stress. From a study of various reinforced concrete slabs designed by the Pennsylvania Railroad it was found that an economical design was obtained by placing 25 per cent of the reinforcing steel in the compression flange and 75 per cent in the tension flange; and this distribution of steel was used for the design of these slabs. Square twisted rods were used for reinforcement, and it was specified that one complete turn was to be made in a length of 15 diameters. It was further specified that the rods be securely wired together at intersections. Stirrups were used, but they were not proportioned to take any stress, being used merely to secure a uniform spacing of the rods and to hold them in placing during the pouring of the concrete. The main tension rods were bent up at the ends in the form of parabolas. The concrete used for the slabs was mixed in the proportions 1:2:4, using washed gravel ¾-in. and under. The actual shear at the ends of the slabs was only about 60 lbs. per square inch.

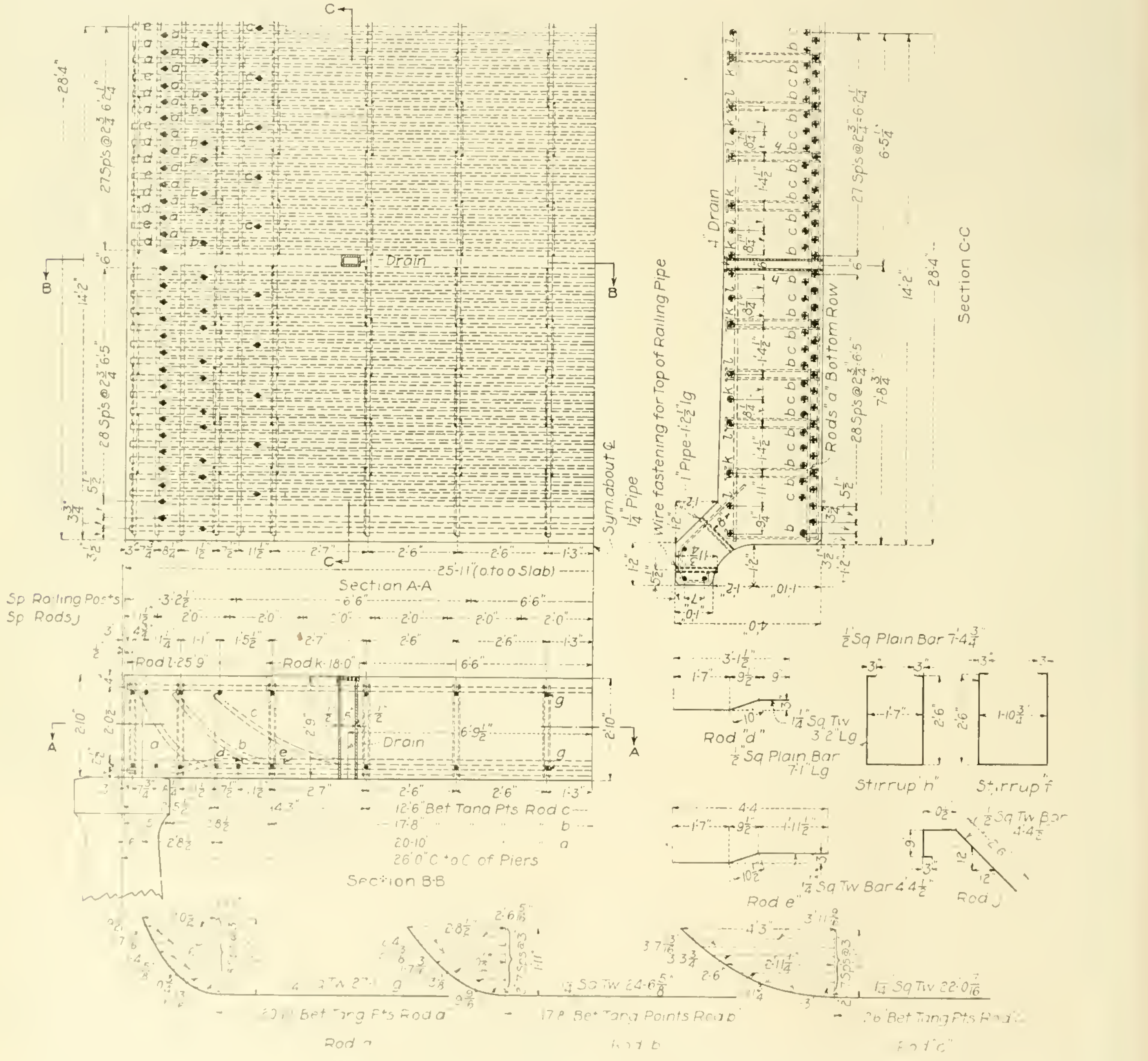

C
B
B
C
A
A
27 Sps @ $2\frac{3}{4}$" 6'-$2\frac{1}{4}$"
28 Sps @ $2\frac{3}{4}$" 6'-5"
28'-4"
14'-2"
Drain
25'-11" (o.to o Slab)
Section A-A
Sp Railing Posts
Sp Rods
Rod l-25'-9"
Rod k-18'-0"
Drain
12'-6" Bet Tang Pts Rod c
17'-8" b
20'-10" a
26'-0" C to C of Piers
Section B-B
Sym. about ℄
$1\frac{1}{4}$" Pipe
Wire fastening for Top of Railing Pipe
1" Pipe-1'-$2\frac{1}{2}$" lg
½" Drain
Rods "a" Bottom Row
27 Sps @ $2\frac{3}{4}$"=6'-$2\frac{1}{4}$"
28 Sps @ $2\frac{3}{4}$" 6'-5"
Section C-C
½ Sq Plain Bar 7'-4¾
Rod "d"
$1\frac{1}{4}$ Sq Tw 3'2" Lg
½ Sq Plain Bar 7'-1" Lg
Stirrup "h"
Stirrup "f"
$1\frac{1}{4}$ Sq Tw Bar 4'-4½"
Rod e
½ Sq Tw Bar 4'-4½
Rod j
20'-10" Bet Tang Pts Rod a
Rod a
$1\frac{1}{4}$ Sq Tw 24-6⅝"
17'-8" Bet Tang Points Rod b
Rod b
$1\frac{1}{4}$ Sq Tw 22'-0$\frac{1}{16}$
12'-6" Bet Tang Pts Rod c
Rod "c"

Fig. 2. Part Plan and Sections of the Reinforced Concrete Slabs Showing Type of Construction, Dimensions and Arrangement of Reinforcement—Bush and Gunpowder River Bridges.

Figure 2 shows a part plan and sections of the reinforced concrete slabs. This drawing shows the dimensions, the arrangement of the reinforcement, and the curves to which the tension rods were accurately bent. All of the main reinforcing rods are 1¼-in. square twisted steel. The compression bars are all straight, the top layer being located 4 ins. below the top of the slab; the tension rods are bent as shown in Section B-B, so as best to resist actual stresses. The rods were quite easily bent to the desired curves by fixing them at the point of tangency and bending them around several pins, the resulting curve being almost exactly that of a parabola. The vertical stirrups are made of ½-in. square rods, bent as shown in Fig. 2. The bottom parts of the ends of the slabs are reinforced with short rods, bent as shown in Section B-B and in the detail.

Each slab is drained by means of four rectangular holes through the slab, placed as shown in Fig. 2. The slab is sloped from the edges and center toward these holes, the difference in elevation being 1 in.
take any stress, being used merely to secure

The slabs are provided with fascias, which project upward and outward 14 ins. from the main slab and serve to hold the ballast in place. Vertical 1¼-in. diameter pipes are placed in these fascias to furnish connections for the railing, and diagonal 1-in. diameter pipes are inserted to provide connections for the conduits, which are suspended from the curb.

Construction.—Each slab was cast in place on the piers in a single piece, and is held in position by dowels set into the piers, except at expansion joints. The 1-in. space between the slabs was filled with tarred paper. In the foreground of Fig. 3 there is shown some of the contractor's equipment for handling materials and for spouting the concrete for the slabs. The forms for several slabs and the steel truss supports for them are shown in place. In the background can be seen derricks and a hoisting and spouting tower, which are being used to construct the piers.

BASCULE SPAN.

Figure 4 shows an elevation of the bascule span and adjacent spans, the type of structure used for the bascule span, and the method of operation being indicated by the drawing. The clear opening of this span is 35 ft., as the girder swings into a vertical position. It is operated by hand through a rack and chain of gears. Mechanism is provided for raising the ends of the lift span and for freeing it from the operating machinery when the span is closed.

The counterweight consists of a steel box incased in concrete. This box is filled with a composition consisting of concrete and steel. The weight of this counterweight is 242 lbs. per cubic foot, being made up of the following proportions: 30 per cent steel at 480 lbs. = 144 lbs. per cubic foot; and 70 per cent concrete at 140 lbs. = 98 lbs. per cubic foot.

The floor construction for the lift span consists of built-up floorbeams and stringers supporting the ties. The fixed span, which carries the operating machinery, has a trough floor, except along the center of the span where there is a 2-in. board walk, 2 ft. 10 ins. wide. The trough floor is filled with concrete, upon which is placed five layers of Hydrox felt.

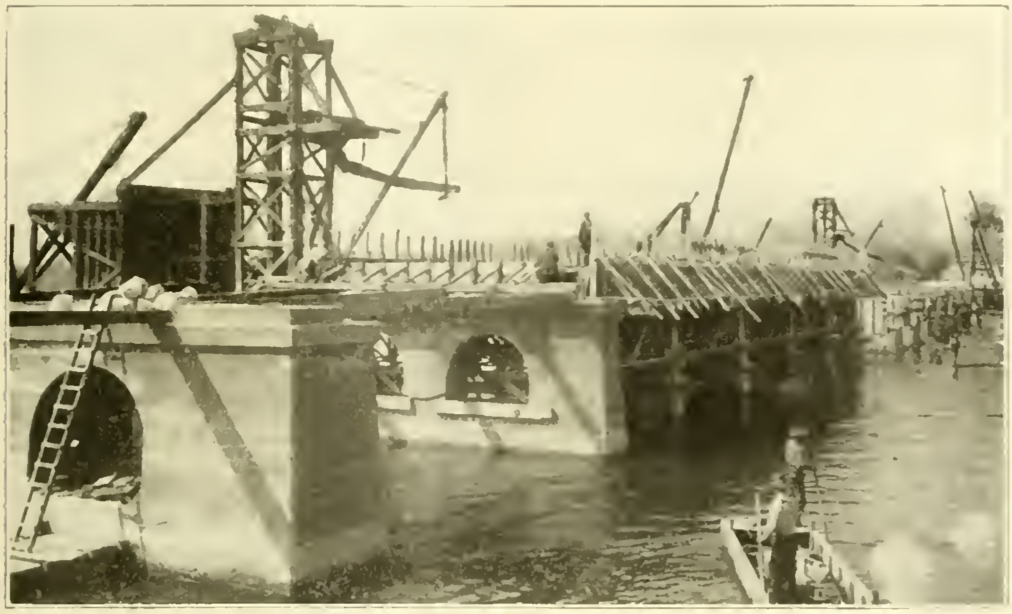


Fig. 3. View Showing Gunpowder River Bridge During Construction; Formwork for Slabs Shown in Place in Foreground; Piers Being Constructed in Background.

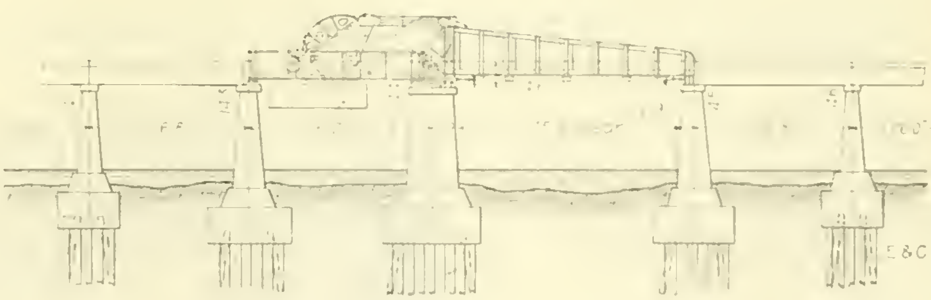


Fig. 4. Elevation of Lift and Adjacent Fixed Spans Showing Type of Construction and Method of Operation—Gunpowder River Bridge.

THE PIERS.

The Design.—In the Gunpowder River bridge there are 186 duplicate piers, two rest piers for the movable span, one center pier for the movable span and two abutments. These piers are constructed of reinforced concrete with pile foundations. Figure 5 shows a plan, elevation and section of the typical piers. The drawing shows the shape and dimensions of the piers, together with the size and position of the reinforcing rods. It also shows the position of the dowels, which were placed in the tops of the piers. It will be noted that some of the piles are placed at an inclination to resist the longitudinal and lateral thrust. Two arched openings were left in the pier shafts to reduce the amount of concrete required. The footings of the piers, which are 10x25 ft. in plan and 6 ft. high, were built of 1:2:4 concrete, to which was added 20 per cent of cement to compensate for loss caused by depositing the concrete under water. The pedestals and pier shafts were constructed of 1:2½:5 concrete.

Piers Nos. 143 and 145, which are adjacent to the center pier of the lift span, are in form similar to the typical piers shown in Fig. 5. In these piers, however, the diagonal reinforcement near the openings is omitted, and the general dimensions are increased. The dimensions of the footings are 13 ft. 5 ins. x 42 ft. 10 ins. x 6 ft. high, and the total height of the piers is 26 ft. The overall dimensions of the top of the piers are 4x36 ft. or, allowing for the 4-in. wash, are 3 ft. 4 ins. x 35 ft. 4 ins. The pier shafts have a batter on all four sides of ¾ in. in 1 ft. The size and distance between the openings in the piers are the same as shown in Fig. 5.

The center pier (No. 144), which carries the bascule span, is similar to the adjacent piers Nos. 143 and 145, except as to size. The dimensions of the footing, which is supported on 113 piles, are 16 ft. 6 ins. x 50 ft. x 6 ft. high. The total height of the pier is 25 ft. 6 ins. The overall dimensions of the top of the center pier are 7 ft. 6 ins. x 41 ft., with a 4-in. wash.

Figure 6 shows a plan, elevation and section of the abutments. It will be noted that these abutments have solid shafts and contain no vertical reinforcement.

Construction.—In building the piers cofferdams were constructed as shown in Fig. 7. The soil was excavated to the depth required for the footing, and by means of bottom-dump buckets concrete was deposited under water around the tops of the piles. The horizontal reinforcement for the top of the footing was then placed under water, and the concreting continued until the top of the footing was reached. The concrete was allowed to set, after which the water was pumped from the caisson, and the pedestal and pier were then constructed in the dry. To insure a good bond between the footing and the pedestal, the top of the fonting was left in a rough condition. Figure 7 also shows the pipe line used for pumping out the caissons and some of the construction equipment.

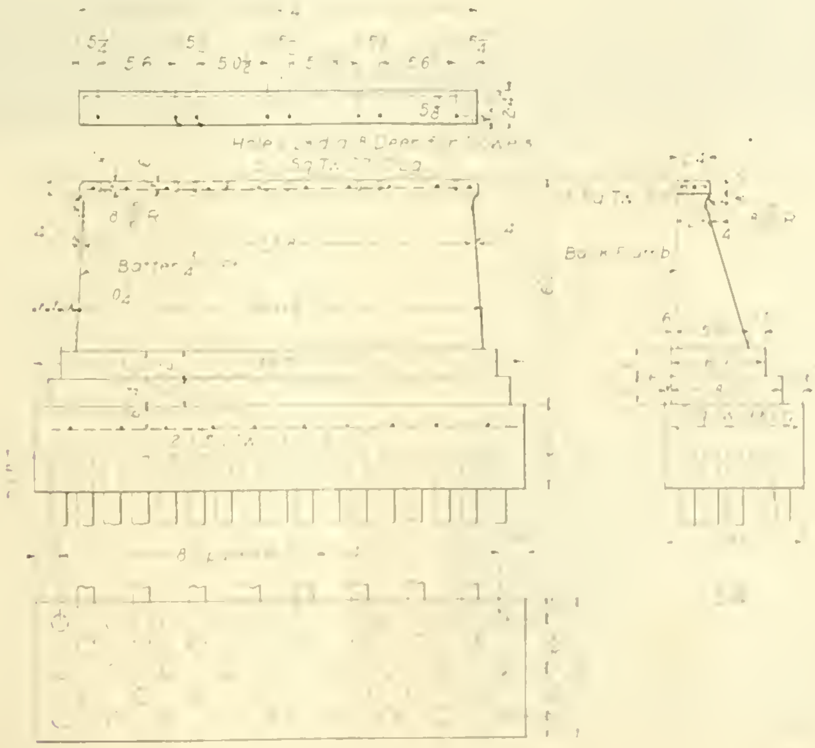


Fig. 5. Plan, Elevation and Section of Typical Piers Showing Type of Construction—Gunpowder River Bridge.

Figure 8 shows a view of an abutment and several of the spans, after the concrete work was completed.

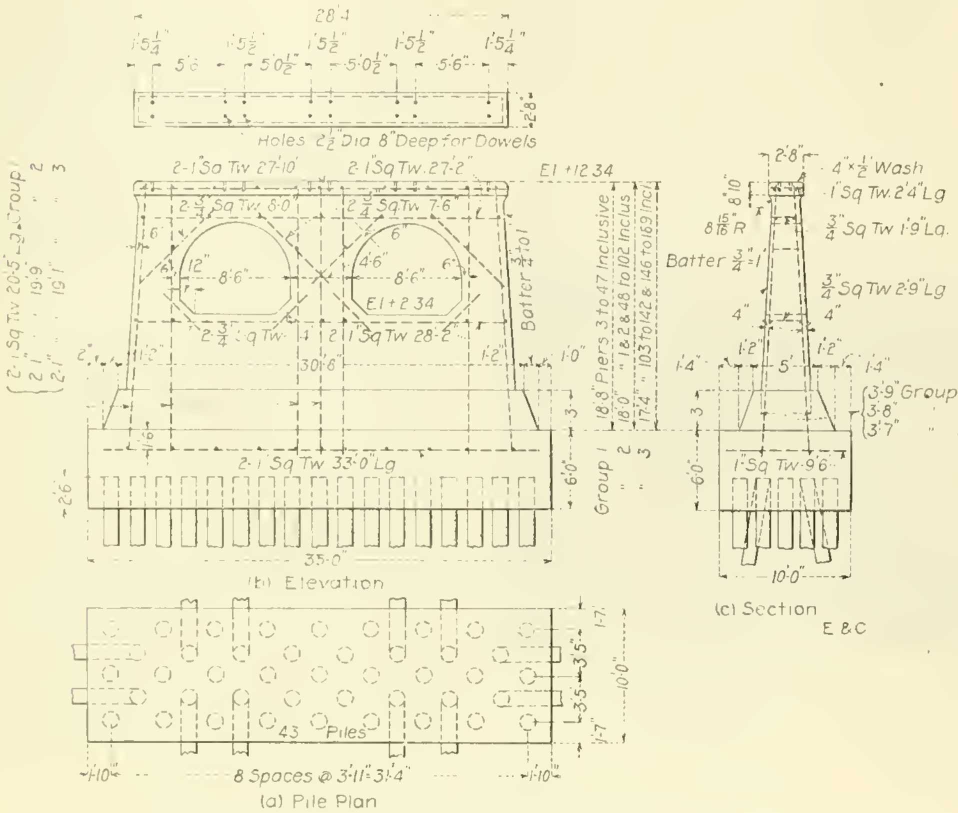


Fig. 6. Plan, Elevation and Section of Abutments Showing Type of Construction—Gunpowder River Bridge.

These bridges were designed by the engineering department of the Pennsylvania Railroad: Mr. A. C. Shand, chief engineer; Mr. H. R. Leonard, engineer of bridges and buildings, and Mr. C. W. Thorn, assistant engineer in charge of construction. The contractors were the Braun & Stuart Co., and Henry Steers, Inc.

Irrigation for Oklahoma.—Mr. Howard V. Hinckley in a recent paper read at the annual convention of the Oklahoma Engineering Society stated that the problem of irrigation for Oklahoma would not be solved by government projects but (in general) by individual pumping plants which could be put in at an expense of $10 to $30 an acre and would furnish water at 50c to $2 an acre-foot. Onions, potatoes or alfalfa would pay the whole cost of the plant the first year and a dividend besides. A wide awake state engineer could do much to assist in developing the state's greatest asset. "Other commonwealths," he said, "have state engineers. Our state has none."

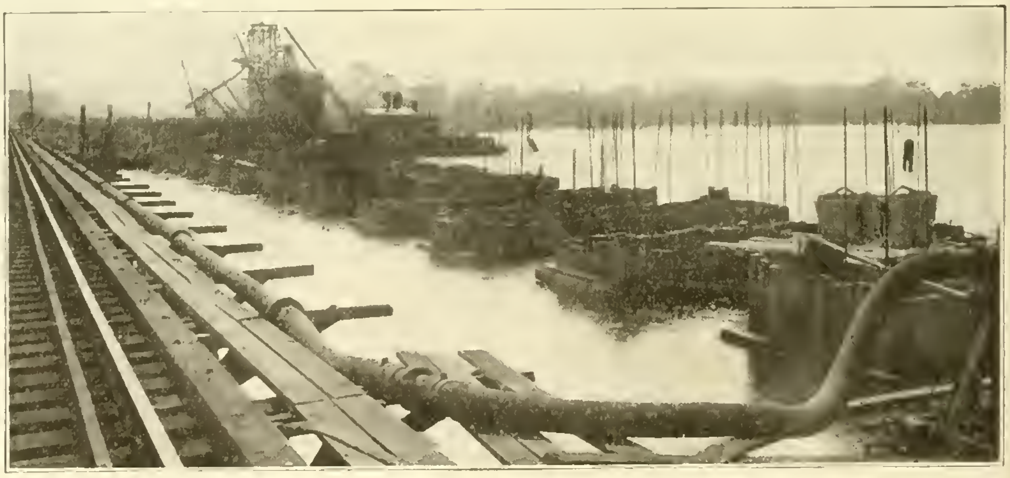


Fig. 7. View Taken During Construction of Footings and Piers Showing Pipe Line and Construction Equipment—Gunpowder River Bridge.

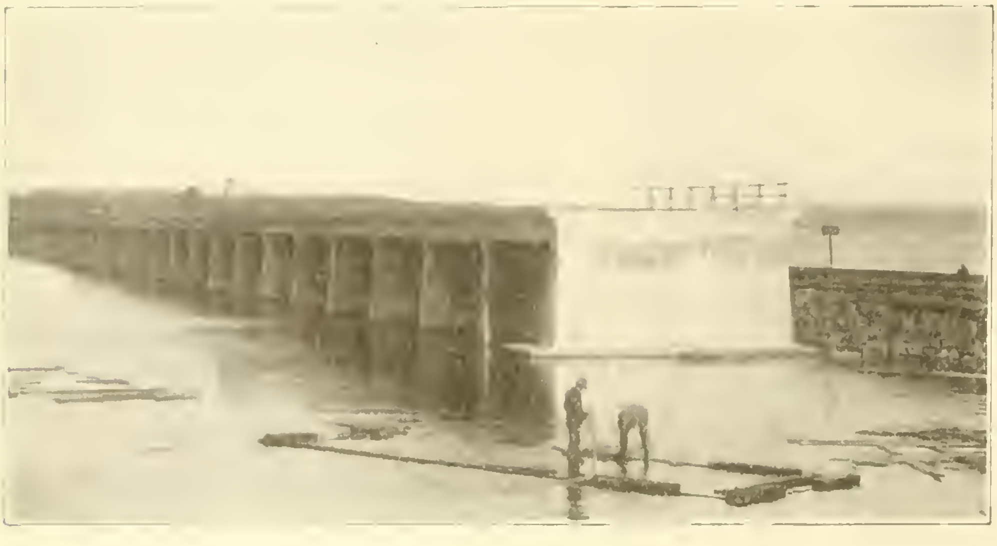


Fig. 8. View Showing Portion of Gunpowder River Bridge—Concrete Work Is Completed.

Specification for Unit Stress in End Posts of Railroad Bridges.

There is little doubt that the tendency to accept some of our so-called standard specifications as law, without making each set of specifications apply directly to a particular piece of work, has resulted in the repetition of certain clauses in practically all specifications. In some cases these clauses have little meaning when applied to the particular piece of work, while in other cases they do not insure a structure of uniform strength in all its parts. The following comments on a commonly accepted clause which is found in practically all railroad bridge specifications were made by Mr. Albert Smith in a paper presented before the recent annual meeting of the Indiana Engineering Society:

The American Railway Engineering Association's specification for bridge members subjected to wind stresses in addition to live and dead load stresses is that "the unit stress may be increased 25 per cent over those given above; but the section shall not be less than required fo live and dead loads and centrifugal force." The specification of the American Bridge Co. is similar, and as this is a standard specification, the writer wishes to point out the inequality thus produced between the strength of the end posts and that of the top chords.

In the top chords the wind stress which is to be combined with the dead and live load stresses is generally very small. Even for very long spans the chord stresses in the top lateral truss due to a wind load of 200 lbs. per lineal foot are not very large. For example, in a span consisting of eight 25-ft. panels, with trusses spaced 18 ft. on centers, the wind load stress in the chord would be 31,250 lbs. If to this we add the overturning effect of 500 lbs. per lineal foot, acting 12 ft. above the rail, and assume that the height of the truss is 35 ft. we find a top chord stress of $47{,}620 + 31{,}250 = 78{,}870$ lbs. In such a truss, with an E-50 loading, this stress would be less than 10 per cent of the combined dead, live and impact stresses in the top chord.

The end post receives a direct stress due to the wind, which is as large or nearly as large as that in the upper chord, and in addition receives a very large unit stress from the bending due to the wind. The resulting unit stresses are likely to exceed 25 per cent of the dead and live load stresses in bridges whose portal braces are not deep.

If the unit stress due to wind is just equal to 25 per cent of the dead and live load stresses no increase will be made in the section. Under the most unfavorable combination of stresses, calling the unit stress in the end post 125 and that in the top chords 110 and calling the ultimate strength of each member U, the relative strength of the two members would be in the ratio of $U/125$ to $U/110$. The end post is therefore 88 per cent as strong as the top chord.

Where the unit stresses in the end post due to wind exceed 25 per cent the addition of a few square inches of steel somewhat improves this ratio.

The actual condition is worse than that shown above when we consider the shape of these members. Both have box sections, with the bottoms latticed. The top chord receives very little bending, while the end post receives a very considerable bending, often sufficient to cause 15 per cent of the total stress from other causes. In resisting this bending the section commonly used for end posts is extremely inefficient. The value *U* in the above ratio represents a lower value in the first term than in the second.

The common answer to objections of this kind is that the wind force will never be as great as that assumed. If we can be certain of this it is obvious that we ought to reduce the amount of the assumed wind loads; if not, let us be as consistent as possible in applying them.

It is always difficult for the writer of specifications to decide as to the proper factor of safety to provide for very remote contingencies. For example, while it may be necessary to design the top lateral bracing for a tornado, if we also design the truss members to resist a tornado while the heaviest live load is passing over the bridge at the rate of speed at which the impact is a maximum, we are providing for a very remote contingency. On the other hand, of all the bridges hit by a tornado, the one which carries a full live load is the one which we most desire to make safe.

The possibility that a fifty-mile-per-hour wind will find the bridge with a full live load is not so remote, and it seems to the writer that if the wind loads are reduced one-half in designing truss members, and the required section is fully supplied, a fairer compromise will be reached.

The Water Supply of Hamburg, Germany.

The average total quantity of water used per day in Hamburg for all purposes in 1912 was 177,998 cu. yds., all of which was supplied from the public water system. Of the above total amount 97.3 per cent was filtered. The population of Hamburg in the year quoted was 986,804, but in 1913 it was 1,006,748. Of the total population, 99 per cent was supplied with water. The foregoing facts have been supplied by the director of the public water works.

Design Features of the Reinforced Concrete Viaduct on Hopple St., Cincinnati, Ohio.

(Staff Article.)

The city of Cincinnati is making an extensive viaduct improvement to eliminate the grade crossings on the line of Hopple St. This street runs in an east-and-west direction, and crosses the railroad tracks at such an angle as to require skewed abutments. This viaduct consists of a 125-ft. earth approach between reinforced concrete retaining walls at the east end; a series of 25 open skew spans and 1 closed span, with a total length of 1,890 ft., each span consisting of two arched reinforced concrete cantilever arms, except the closed span which consists of a single cantilever and a 10 ft. earth approach between reinforced concrete retaining walls at the west end. The structure when completed will pass over Spring Grove Ave., the Baltimore & Ohio Southwestern R. R. tracks, Mill Creek, and the Chicago, Hamilton & Dayton Ry. tracks. All of the piers, with the exception of two piers in the channel of Mill Creek and four piers near the west end of the viaduct, have concrete pile foundations; those not having concrete pile foundations rest on shale. The material at the surface of the ground generally consists of yellow clay, followed by strata of sandy clay and gravel, blue clay, soapstone and shale. The viaduct spans have varying lengths of 65, 70, 75 and 80 ft., each span consisting of two equal or unequal arched cantilevers. Starting at the east end of the viaduct the first 905 ft. are on a 2.70 per cent up grade; the next 515 ft. are on a 0.56 up grade, and the west 635 ft. are on a 2.70 down grade. The structure has an extreme width of 60 ft., consisting of a 46-ft. roadway and two 7-ft. sidewalks. The roadway construction consists of reinforced concrete slabs on longitudinal girders and cross beams, supporting a wood block pavement. In designing the structure provision has been made for future street car lines. The sidewalks extend 4 ft.

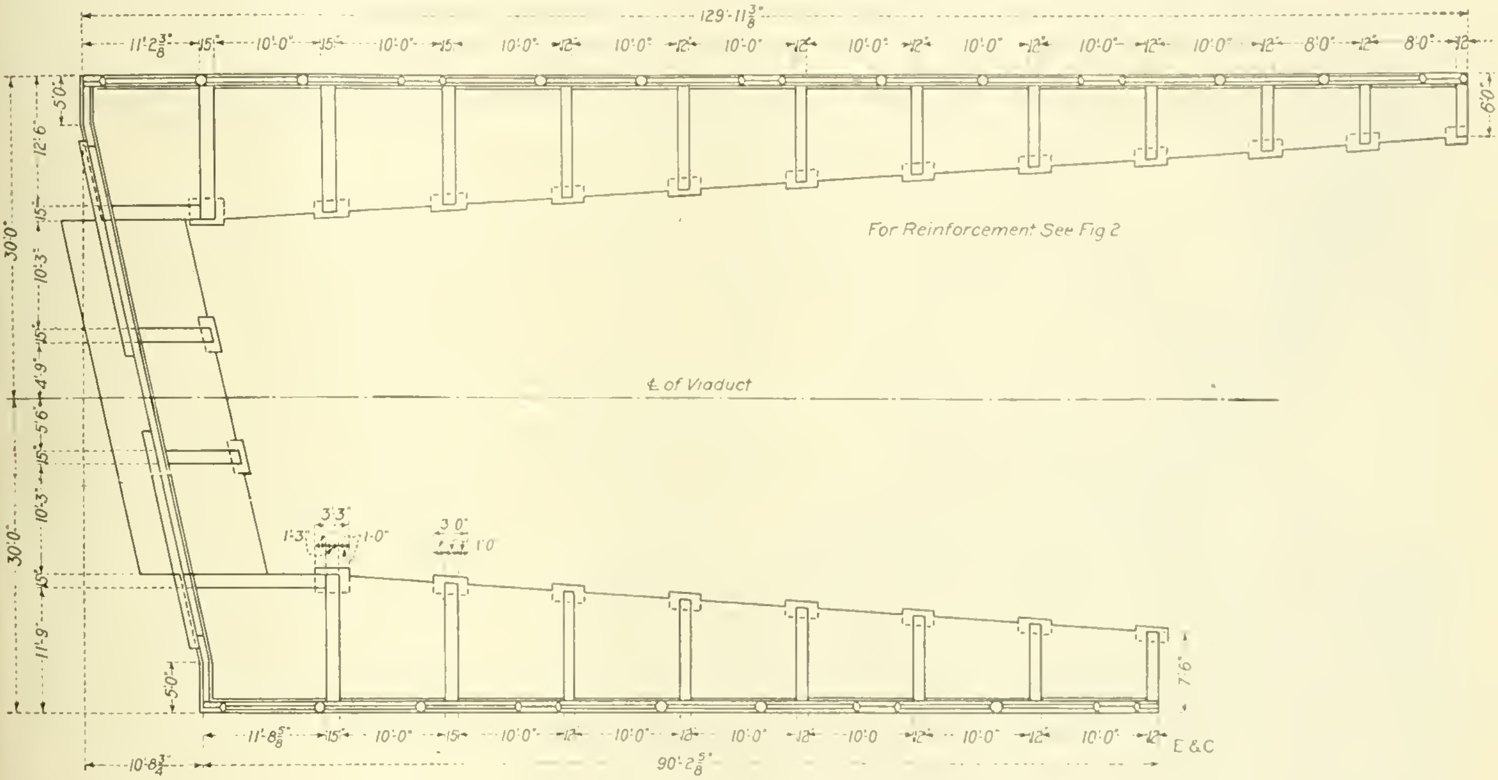
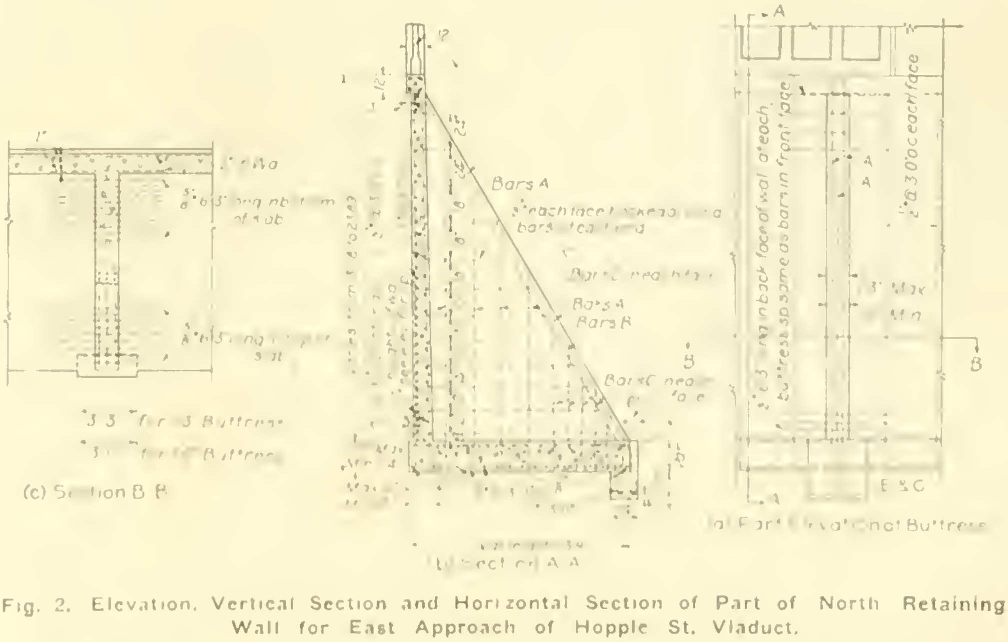


Fig. 1. Plan of Retaining Walls for East Approach of Hopple St. Viaduct, Cincinnati, O.



(c) Section B B

(b) Section A A

(a) Part Elevation of Buttress

Fig. 2. Elevation, Vertical Section and Horizontal Section of Part of North Retaining Wall for East Approach of Hopple St. Viaduct.

beyond the outside longitudinal girders. A reinforced concrete stairway connects the viaduct with Spring Grove Ave., over which it passes. The outside longitudinal girders are paneled, and the viaduct is provided with ornamental concrete railings. The contract price of the structure is $113,765. The principal design features of the east and west approaches will be considered in this issue. The design of the superstructure and substructure of the viaduct spans will be treated in subsequent issues.

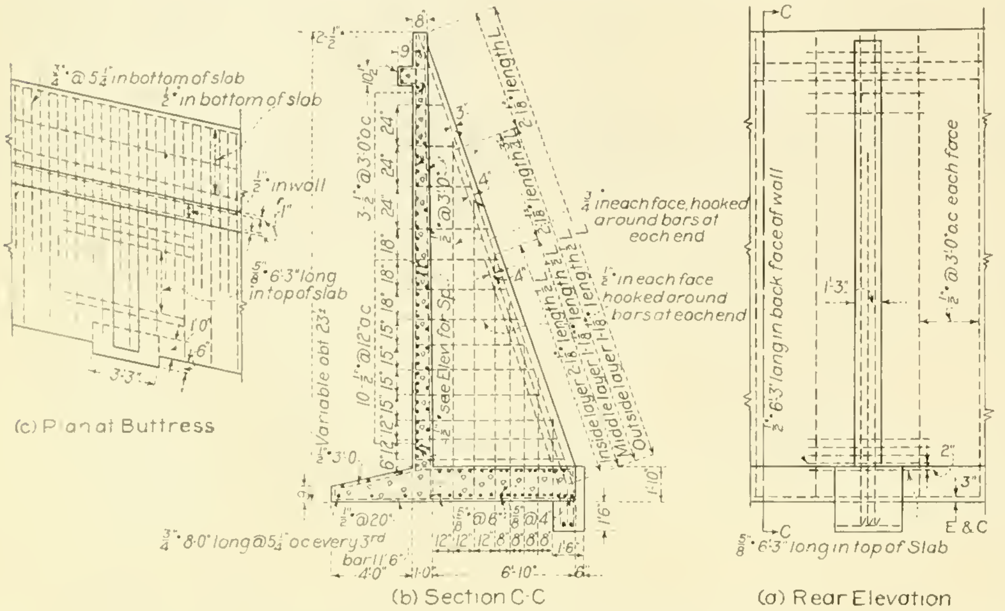

¾" @ 5¼" in bottom of slab
½" in bottom of slab
½" in wall
⅝" 6'3" long in top of slab
(c) Plan at Buttress
¾" 8'0" long @ 5¼" oc every 3rd bar 11'6"
¾" in each face, hooked around bars at each end
½" in each face hooked around bars at each end
(b) Section C-C
⅝" 6'3" long in back face of wall
½" @ 3'0" oc each face
⅝" 6'3" long in top of Slab
(a) Rear Elevation
E & C

Fig. 3. Elevation, Vertical Section and Plan of Part of West Retaining Wall for East Approach of Hopple St. Viaduct.

I.

EAST APPROACH.

The east approach consists of an earth fill confined between reinforced concrete retaining walls. Figure 1 shows a plan of the end and side retaining walls. The side walls are of unequal length, the north wall being 129 ft. 11⅜ ins. long and the south wall 90 ft. 2⅝ ins.; these retaining walls are spaced 60 ft. apart, out to out. The height of the north wall at its west end is 23.89 ft., from the bottom of the spread footing at elevation 65.70 ft. to the top of the retaining wall at elevation 89.59 ft. The height of this wall at its east end is 13.08 ft., from the bottom of the footing at elevation 73.00 ft. to the top of the wall at elevation 86.08 ft. By noting the elevations given for the top and bottom of the wall at each end, it will be seen that both the top and bottom of the wall are on a slope. The heights given do not include the concrete railing. The width of the footing for the north retaining wall at its west end is 13 ft. 9 ins., and at its east end the width is 6 ft. 0 ins.; the corresponding dimensions for the south retaining wall are 13 ft. 0 ins. and 7 ft. 6 ins. The buttresses for the side walls are spaced 11 ft. on centers, except for the two panels at the east end which have a 9-ft. spacing, and the end panels near the west end where the panel length is slightly increased due to the greater thickness of the buttresses. The spacing of the buttresses for the west retaining wall is 11 ft. 6 ins. on centers. The thickness of the reinforced concrete buttresses for the west wall and for the west panels of the side walls is 15 ins. The remaining buttresses for the side retaining walls have a thickness of 12 ins.

Figure 2 (a) shows an elevation of a portion of the north retaining wall near a buttress; Fig. 2 (b) shows a vertical section of this wall; and Fig. 2 (c) shows a horizontal section through the wall and buttress. These detail drawings show the type of construction, arrangement and size of the reinforcement, and the dimensions. It will be noted that the coping for this wall is 12 ins. wide, 12 ins. deep and extends 3 ins. beyond the face of the wall. The thickness of the wall at the top is 9 ins., and at the bottom it varies from 10 to 12 ins. By referring to Fig. 2 it will be seen that a small footing is provided at the toe of each buttress, this footing projecting 6 ins. beyond the buttress and 1 ft. 6 ins. below the retaining wall footing, thus affording an increased resistance against sliding. To prevent hydrostatic pressure against the retaining walls weep holes are provided at each buttress, one hole being placed at the top of the footing and another at the elevation of the top of the outside embankment. The side walls have an ornamental reinforced concrete railing. The south retaining wall is similar in design to the north one, except as to height and length.

Figure 3 (a) shows an elevation of a portion of the west, or end, retaining wall which marks the beginning of the arched cantilever spans; Fig. 3 (b) shows a vertical section through the end retaining wall; and Fig. 3 (c) shows a part plan at a buttress. These drawing show the type of construction, dimensions, and size and arrangement of the reinforcement. It will be noted that the end wall differs from the side wall in that the footing extends on both side of the retaining wall (see Figs. 2, b and 3, b). All horizontal reinforcing bars in the end wall extend around the side walls for a distance of 30 ins. The thickness of the wall at the top is 8 ins., and at the bottom the thickness is 12 ins.; its height at the center of the wall is about 23 ft. Weep holes are provided in the end wall near each buttress, at the top of the footing and at the elevation of the outside ground line.

WEST APPROACH.

The west approach, which has a length of 40 ft., has its end wall on a skew of 30° 40' (see Fig. 4, a). The type of construction used for the end wall is similar to that used for the end wall of the east approach. Its height at the center of the roadway, from base of footing to top of wall, is about 24.4 ft. The side retaining walls, which have a length of about 37 ft. 5 ins., have no buttresses and are built in two parts, separated by a tar paper joint, the division being in a horizontal plane at the elevation of the spring line of the adjacent arched cantilever span. Figure 4 (a) shows a plan of the portion of the side wall below the horizontal joint. The portion above the joint is paneled and carries an ornamental concrete railing. By referring to Figs. 4 (a) and 4 (b) it will be noted that a 14-ft. section of this wall adjacent to the pier, has no footing. The west 23 ft. 5 ins. of the wall have a footing which projects 1 ft. 9 ins. inside and 2 ft. 9 ins. outside of the wall, as shown in Fig. 4 (c). Weep holes are provided at each buttress in the end wall; but they are not required in the side walls, as the buttresses and parts of the footings are omitted.

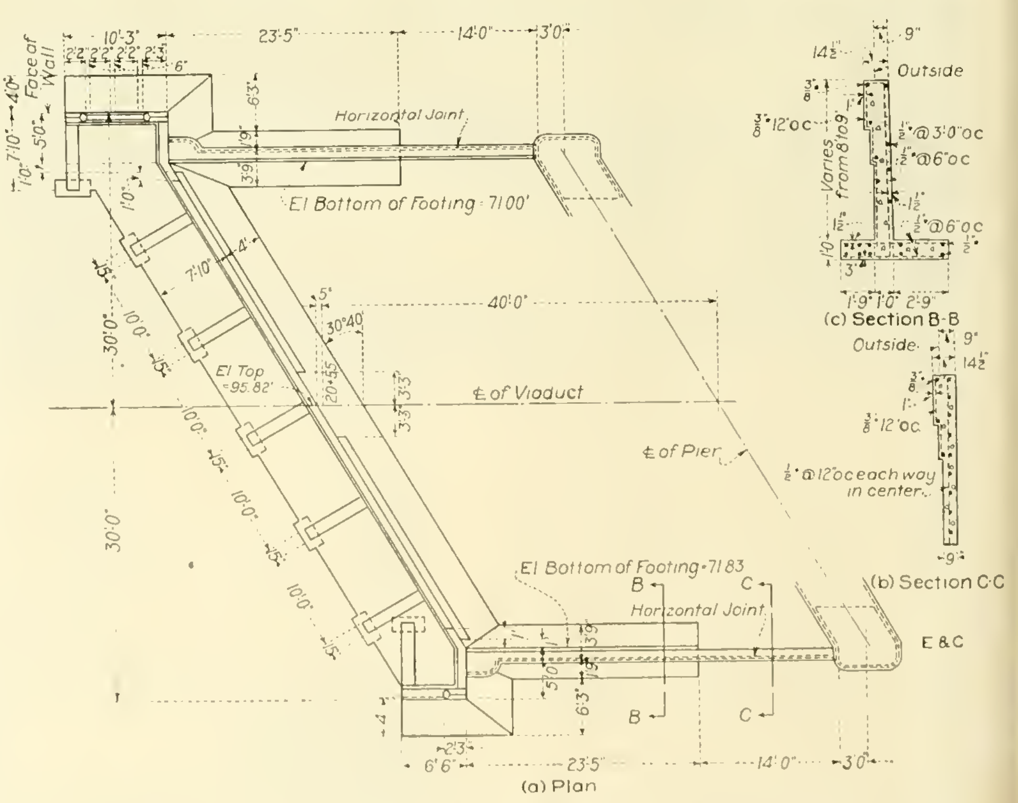


Fig. 4. Plan and Vertical Sections of Retaining Walls for West Approach of Hopple St. Viaduct.

A new system of road designation for the convenience of tourists has been adopted in France. Every road in the country will be given a name and a number, and these designations will be painted upon direction posts at the road crossings and the one-hundred-meter posts along the roads. The tourist starting on a journey will need only a strip of figures, and he will be able to find his way anywhere.

Sewerage

Methods and Cost of Constructing Large Brick and Concrete Sewers in Chicago, with Notes on the Cost Keeping System Employed.

The old-time sewer contractor is said to have prepared his bid on special assessment work, by first finding out the amount of the assessment, and then spreading this figure over the work, to bring the bid up to the assessment or somewhat under.

Although the assessment is based on an estimate made by the engineer, yet the contractor, by this method, is accepting the figure of another on cost of work, work about which he should have a definite and precise knowledge. The average sewer contractor apparently spends little time on cost analysis. He can tell you the total cost of a piece of work after it is completed, but the distribution of that cost among the different items is not made as the work progresses. It is rather difficult to build up an estimate, knowing only the total cost of other pieces of work. Unit costs are needed, and unit costs cannot be obtained except by a system of daily reports showing distribution of labor.

The construction methods employed in building large concrete and brick sewers in Chicago, the corresponding unit construction costs, and the cost keeping system employed, were described and stated in a recent very valuable paper by Mr. H. R. Abbott, of the engineering department of the Sanitary District of Chicago, before the Illinois Society of Engineers and Surveyors. The paper, practically in full, follows:

Before detailing the particular sewer work, I would point out that with the exception of very small stretches, all of the work described in this paper is built in good stiff blue clay, in the Sanitary District of Chicago.

WEST 39TH STREET CONDUIT.

This conduit was built in W. 39th St., from Western Ave. to Robey St.; thence south to the west arm of the south fork of the Chicago River. The total length was 2,346 ft., of which 1,868 ft., was plain concrete, a section of which is here shown as Fig. 1, and 478 ft. reinforced concrete, the reinforced section being under railroad property. It is 12x11 ft. in size, of elliptical section. Connection was made at Western Ave. with a conduit of similar section, extending north in Western Ave. to the main channel of the Sanitary District. This section was built by the City of Chicago. The conduit was designed to discharge from 100 to 200 cu. ft. per second. The invert at the intake at the west arm is placed at elevation —11.77 Chicago city datum, and the conduit has a grade of 1 ft. in 10,000 ft. The flow line is about 3 ft. below crown.

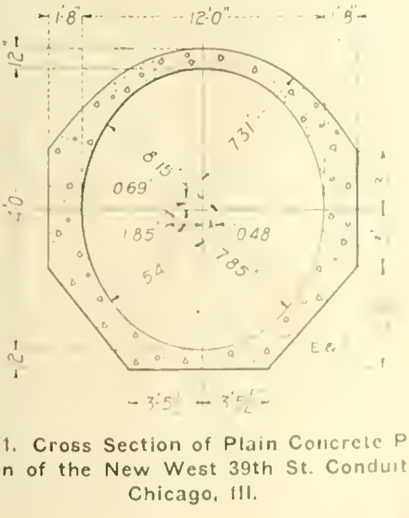


Fig. 1. Cross Section of Plain Concrete Portion of the New West 39th St. Conduit, Chicago, Ill.

Excavation.—Excavation was started at the Western Ave. end in open cut. A Bucyrus 70-ton steam shovel was used with a 1¾ cu. yd. dipper. The shovel was mounted on five 16x18-in. timbers, 30 ft. long, with two 2-in. truss rods to each timber. The top 4 ft. of trench was excavated about 3 ft. wider than the outside lines of the masonry, since no bracing was put in near the top of the trench. Below this the trench excavation was made to the exact width of the masonry, plus an allowance of 4 ins. for sheeting. Although a variation in and out was unavoidable, it did not exceed 2 ins. in either direction. The trench width was 15 ft. 8 ins.; average cut was 23 ft. 6 ins., making an excavation of 13.7 cu. yds. per running foot. On account of the deep cut, the shovel was equipped with a 36-ft. boom and a 54-ft. dipper handle. As there was liabilty of slides and cave-ins, the excavation was handled in two lifts. On the first run the shovel excavated the top 10 ft., using 9-ft. sheeting with one set of bracing placed about 6 ft. below the ground surface. The shovel dug ahead of the finished cut from 75 to 100 ft., then backed up and excavated the lower 13½ ft. The lower lift was taken out between steel beams, each built up of two 10-in. I-beams with cover plates, 30 ft. long, held in place by screw braces set 7 ft. back from each end. This replaces the ordinary wooden bracing and allows a free movement of the dipper in the trench for three moves or 36 ft. When a section is finished the beams are carried ahead by the dipper, the wooden braces are replaced on the top sheeting and another set of 9-ft. sheeting is placed with two sets of braces for the lower portion of the trench, the lower end of the sheeting being at a point where the invert curve meets the side wall. The lower sheeting back of the concrete was left in permanently. The bottom was trimmed and shaped by four or five bottom men, the material being cast ahead where the shovel could reach it. An iron frame or template built to the dimensions of the outside lines of the masonry was set up every 12 ft. as a guide in trimming the sides. The excavated material was loaded direct from the shovel on to 4-cu. yd. dump cars operating on a 3-ft. gage track. Ordinarily, the upper lift made the backfill, and the lower lift was run to a spoil area in McKinley Park, a haul of about ⅜ mile. The sheeting was 2x10 in. hemlock, the braces 8x8 in. and 6x6 in., with stringers 6x8 in. of yellow pine.

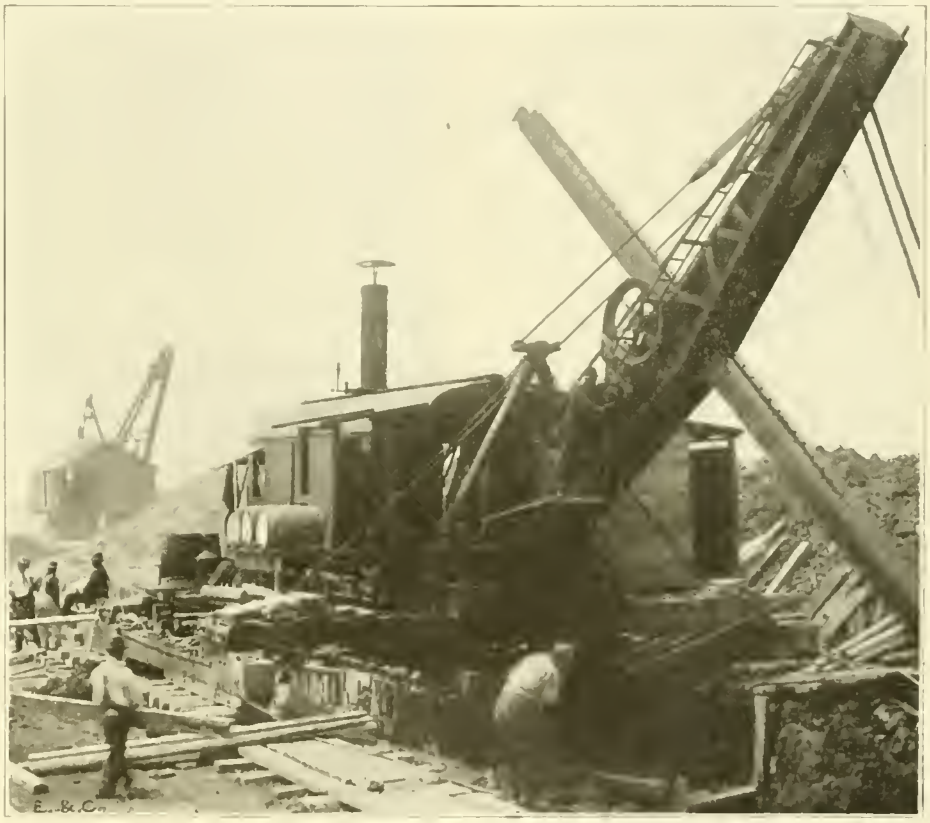


Fig. 2. View of Excavating and Backfilling Plants at Work on the South 52d Ave. Sewer, Chicago, Ill.

Concrete.—The concrete mixer was mounted on timbers to span the trench. A No. 2 Chicago mixer, holding 25 cu. ft. of dry material, was used. Adjustable spouts were used for pouring the concrete, the spout-man standing on braces in the trench and deflecting the concrete to any point required. The pouring was made in three runs, each usually being about 16 ft. long. The first or dish extended to 2 ft. above the bottom of the

TABLE I.—UNIT COST OF CONSTRUCTING THE PLAIN CONCRETE SECTION OF THE WEST 39TH ST. CONDUIT—SIZE 12x11 FT.—AVG. CUT, 23 FT. 6 INS.

Item	Cost Per lin. ft.	Cost Per cu. yd.
Excavation, labor	$ 2.53	$0.188
Excavation, plant	0.64	0.046
Backfill	0.86	0.113
Waste disposal	0.89	0.120
Miscellaneous	0.7[illegible]	
Coal	1.21	
Lumber	0.99	
Concrete masonry	10.1[illegible]	
Labor		1.315
Cement		1.075
Sand		0.576
Gravel		1.103
Plant		0.081
		$4.15

Total cost per lineal foot, $18.29.
Cost percentages: For material and plant, 84 per cent, for labor, 16 per cent.

trench; the second, or sides, extended 2 ft. above the springing line; the third or arch completing the section. The invert was shaped up with a wooden template or bulkhead, conforming to the inside and outside lines of the masonry, on which the forms were placed after the concrete was set. The forms were built up of 2x6-in. lagging, laid on 6-in. channels, bent to shape. At the springing line, a 6x6-in. timber rested on angles bolted to the channel, being held in place by a ⅞-in. pin running through both timber and angle. After the sides were poured and set, the braces were removed and the lagging placed for the crown. The channels for the arch were reinforced with two plates.

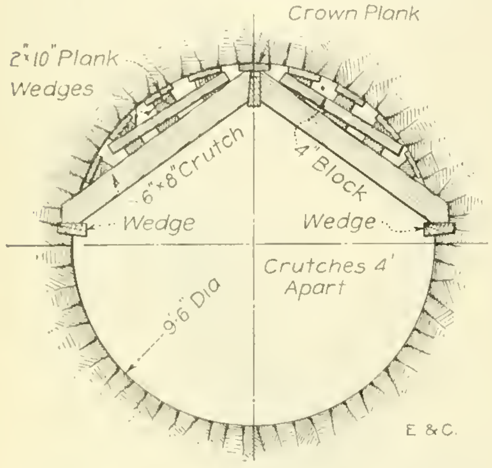


Fig. 3. Detail of Timbering in Place to Support Roof in Tunnel Section of So. 52nd Ave. Sewer.

No manholes were built and no lateral connections were made, but 24-in. tile were set in the arch at intervals for future connections. The contract specified a concrete composed of 1 part Portland cement, 3 parts sand, and 5 parts crushed stone or gravel; the engineer, under the specifications, having the right to vary the proportions of fine and coarse aggregate, but maintaining the proportion of 1 part cement and 8 parts aggregate. Gravel proved very satisfactory. The mix was fairly wet, except on the crown of the arch, where a dry mix was necessary to prevent the concrete running.

The average progress per day of 9 hours was 30 ft. for both shovel and mixer, for the plain section. This means 120 cu. yds. of excavation, with disposal in backfill or spoil bank. The actual cost of excavation, backfill, and spoiling can be seen by reference to Table I. The concrete averages 2½ cu. yds. per ft. A daily average of 75 cu. yds. was placed. The average progress per day on the reinforced section was 24 ft. per day, the slowing up being due to the time used in placing the reinforcing steel.

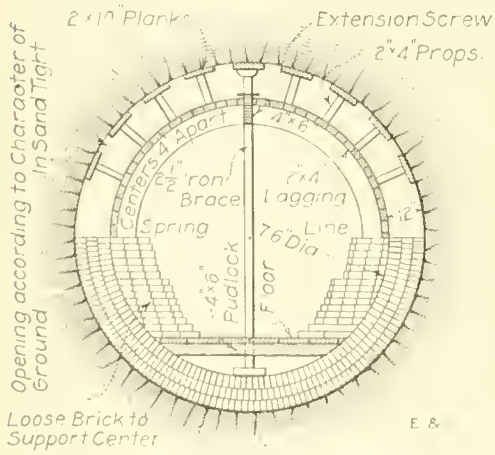


Fig. 4. Tunnel Section of So. 52nd Ave. Sewer Showing Invert Built, Timbering Removed and Centers Set for Arch.

On the same platform with the mixer was mounted a small boom derrick and hoisting engine. This facilitated the removal of stringers and braces, and pulled the mixer platform back and forth. The material for concrete was delivered to a platform laid on the ground alongside of the mixer, being hauled in 4-cu. yd. dump cars by dinky engines an average distance of ⅜ mile.

Reinforced Section.—This section is of the same dimensions as the plain, but was reinforced to strengthen the conduit where it passed under railroad property. The same methods of construction were used as on the plain section. The reinforcing steel averaged 44 lbs. to the cubic yard of concrete.

Backfill.—Backfill was made by the 4-yd. dump cars, track being swung in over the conduit as the filling progresed. Centers were left in until the sides were thoroughly compacted and at least 1 ft. of filling had been placed over the top of the arch. Unit costs on the reinforced concrete portion of this sewer are given in Table III.

SOUTH 52ND AVE. SEWER (CICERO SECTION).

This is a three-ring brick sewer in South 52nd Ave., Cicero, extending from Ogden ditch to 22nd St., a total length of 10,000 ft., of which 7,300 ft. was 7½ ft. and 2,700 ft. was 7 ft. in diameter; 1,050 ft. of the 7½-ft. section was in tunnel.

With the exception of the tunnel the entire sewer was built on the line of an old 4x5-ft. wooden box sewer. The sewage flow was usually held back for periods of 8 to 16 hours, depending on rainfall, by a temporary gate, consisting of an enclosed box 3x3x12 ft., having a sliding door working vertically about 4 ft. from the upstream end. The old wooden box sewer was first uncovered at a point 600 to 1,000 ft. ahead of the steam shovel. After the top was removed, the gate was lowered into the old box and packed in place with sand bags. The gate was operated by a lever at the ground level, by a night watchman, who generally closed the gate at 6 a. m. and opened it at 7 or 8 p. m. A 45-ton Bucyrus steam shovel, equipped with a 1¼-cu. yd. dipper, excavated the trench, placing the excavated material alongside. The average cut was 21 ft. made in a single cut. The existing box sewer was ripped out by the shovel as the trench advanced.

TABLE II.—CONSTRUCTION, FORCE AND RATES OF PAYMENT ON W. 39TH ST. CONDUIT, TO ACCOMPANY COSTS IN TABLE I.

Employee—	Wage per day.
1 superintendent	$8.00
1 shovel engineer	7.00
3 dinkey engineers, each	3.60
1 cranesman	4.50
1 fireman	3.00
3 switchmen, each	2.25
2 flagmen, each	1.75
1 coal passer	2.50
3 foremen, each	4.50
1 hoisting engineer	5.60
4 bottom men, each	3.85
50 to 60 laborers, each	2.50
1 team	5.06
1 carpenter	4.80
1 machinist	3.50
1 machinist's helper	2.50
1 office boy	2.00
1 material man	2.50
1 watchman	2.60
3 water boys, each	1.00

Sheeting 2x10 ins. by 16 ft. long was used with three set of stringers and braces. On about 70 per cent of the work the sheeting and one set of braces and stringers were left in.

During the progress of the work, several severe rainstorms occurred, causing considerable delay and some damage. In a portion of the work where sheeting had been pulled, a severe rain caused the bank to slide, which, together with the added weight of the spoil bank, caused a collapse of 130 ft. of completed sewer. The U. S. Weather Bureau record showed a precipitation of 1.57 ins. for two hours. The cost of repairs for this 130 ft. was $11.46 per ft., or 94 per cent of the first cost. On account of storms and the softening of the bank by storm and ground water shorter lengths collapsed. Because of the nearness to building foundations, thereafter, sheeting and one set of braces was left in place at an additional cost of 90 cts. per running ft. of sewer.

On the unpaved portion of the street the excess excavation was spoiled over the street. On the balance of the work it was loaded directly into wagons by the shovel, although a small portion of the excess was handled by a small Thew revolving steam shovel, loading wagons from the spoil bank.

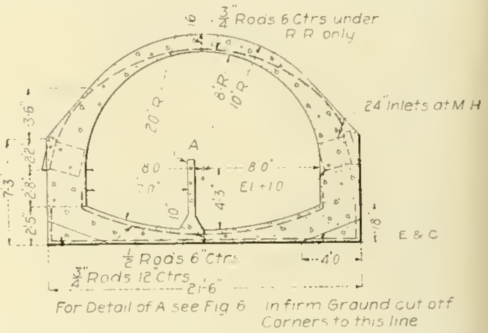

¾" Rods 6" Ctrs under R.R. only
24" Inlets at M.H.
A
8'0"
El +1.0
7'3"
2'5"
2'8"
2'2"
3'6"
½" Rods 6" Ctrs
4'0"
¾" Rods 12" Ctrs
21'6"
E. & C.
For Detail of A see Fig. 6
In firm Ground cut off Corners to this line

Fig. 5. Cross Section of 12x16 ft. Horseshoe Section of So. 52nd Ave. Sewer, Sanitary District Section, Showing Division Wall.

Utica cement was used, proving satisfactory in standing up under the night flow of water over new work. A small amount of pointing-up proved necessary in a number of cases where water was passed over the brick work as soon as laid, and in a special case, when the breaking of the gate had flooded out the bricklayers before the invert could be lai complete. All material was teamed to thc work, the average haul being ¾ mile. Connections were made with all lateral sewer and existing house connections.

TABLE III.—UNIT COST OF CONSTRUCTING THE REINFORCED CONCRETE SECTION OF THE W. 39TH ST. CONDUIT—SIZE, 12x 14 FT. AVG. CUT, 22 FT.

Item—	Cost Per lin. ft.	Per cu. yd.
Excavation, labor	$ 2.43	$0.194
Excavation, plant	0.64	0.046
Backfill	1.24	0.249
Waste disposal	0.31	0.041
Miscellaneous	2.26	.....
Coal	1.21	.....
Lumber	0.99	.....
Concrete masonry	14.85	.....
Labor		1.975
Cement		1.055
Sand		0.576
Gravel		1.103
Reinforcing steel		1.103
Plant		0.084
		$5.896

Total cost per lineal foot, $23.93.

Cost percentages: For material and plant, 53 per cent; for labor, 47 per cent.

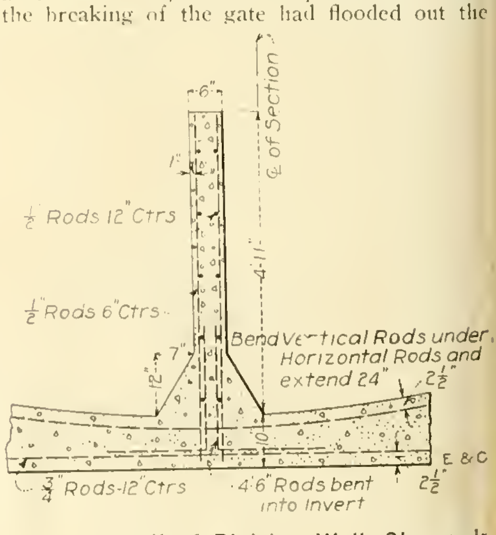


Fig. 6. Detail of Division Wall Shown Ir Fig. 5.

The average progress per day on the 7-ft. section was 45 ft., equivalent to 330 cu. yds. of excavation, while on the 7½-ft. section the average progress was 70 ft. per day, with 20 ft. cut, or 500 cu. yds. of excavation per day. The difference in the progress between these two sections was partly due to the fact that the 7½-ft. sewer was built in a street 80 ft. wide, with open prairie on one side and unlimited room for work, and the 7-ft. section was built in a 66-ft. street with scant open space adjacent to the street. Tables IV and V give the unit costs on the 7 ft. and the 7½ ft. 6-in. sections respectively.

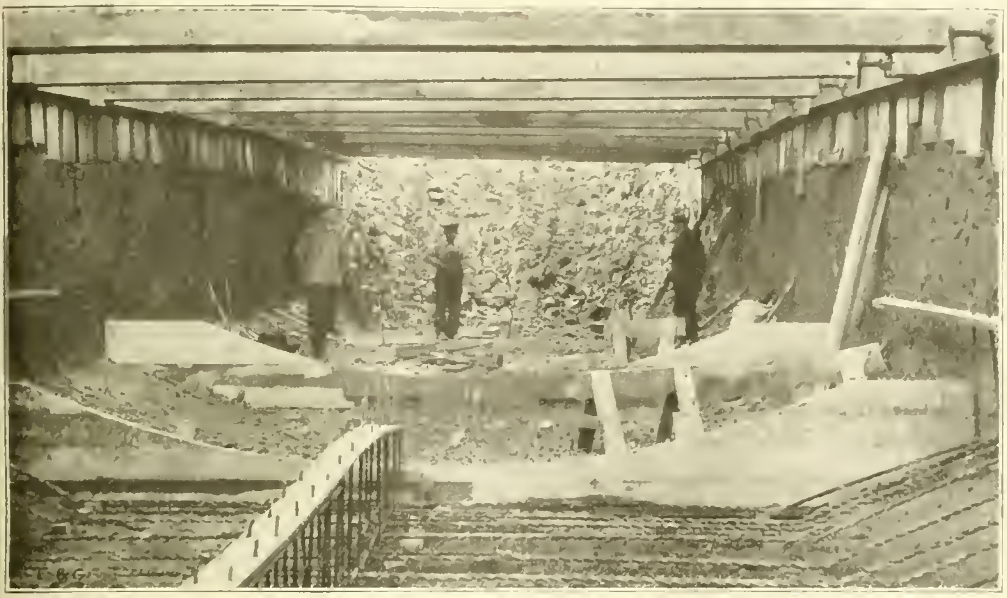
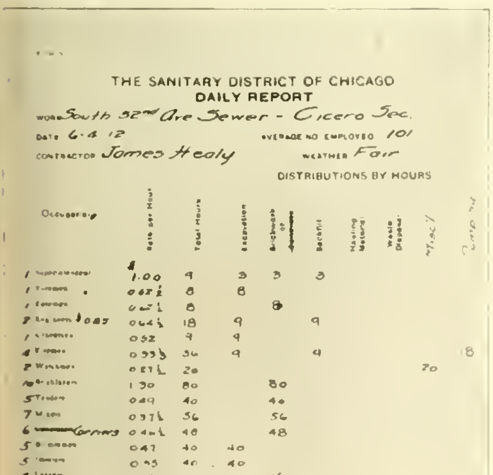


Fig. 7. View of Invert Forms and Reinforcement of 12x16 ft. Horseshoe Section Shown in Fig. 5.



THE SANITARY DISTRICT OF CHICAGO
DAILY REPORT

WORK South 52nd Ave Sewer - Cicero Sec.
DATE 6-4-12 AVERAGE NO. EMPLOYED 101
CONTRACTOR James Healy WEATHER Fair

DISTRIBUTIONS BY HOURS

ig. 8. Specimen Daily Report Form for Reporting Labor Data for Sewer Work, Sanitary District of Chicago.

The average number of brick laid per day per bricklayer was 4,900 in the 7-ft. section and 5,900 in the 7½-ft. sewer. Backfilling was done with a Monaghan revolving derrick, equipped with a Page orange peel bucket, capacity 1 cu. yd. This is a very efficient machine for backfilling, but the operator should avoid dropping the load from any distance, as it is liable to crack the masonry, especially when working during wet weather, when the backfilling is saturated with water. A view of the excavating and backfilling plants is shown in Fig. 2.

Some special items may be worthy of mention, such as the cost of hand excavation in a sewer trench of this size, moving plant, etc. At the Illinois Central R. R., where the sewer passed under the tracks, the excavation was made by hand, loaded into wheelbarrows and wheeled to the edge of the right-of-way, at which point it was handled by the orange-peel derrick.

TABLE IV.—UNIT COSTS OF CONSTRUCTING THE CICERO SECTION OF THE SO. 52ND AVE. 7-FT. BRICK SEWER—AVG. CUT, 21 FT.

Item—	Cost Per lin. ft.	Cost Per cu. yd.
Excavation, labor	$ 2.22	$0.308
Excavation, plant	0.28	0.040
Backfill, labor	0.81	0.226
Backfill, plant	0.14	0.038
Waste disposal	1.00	0.544
Pumping	0.19	.....
Miscellaneous	0.62	.....
Coal	0.35	...
Lumber	1.12	.....
Brick masonry	7.60	.....
Labor		$2.63
Teaming sand and cement		0.49
Brick		1.08
Cement		0.57
Sand		0.29
		$8.06

Total cost per lineal foot, $14.33.

Cost percentages: For material and plant, 49 per cent. For labor, 51 per cent.

TABLE V.—UNIT COSTS OF CONSTRUCTING THE CICERO SECTION OF THE SOUTH 52ND AVE. 7-FT. 6-IN. BRICK SEWER—AVERAGE CUT, 20 FT.

Item.	Cost Per lin. ft.	Cost Per cu. yd.
Excavation, labor	$1.63	$0.226
Excavation, plant	0.28	0.044
Backfill, labor	0.43	0.119
Backfill, plant	0.14	0.038
Waste disposal	0.41	0.417
Pumping	0.15	.
Miscellaneous	0.81	
Coal	0.37	
Lumber	0.47	.
Brick masonry	7.50	
Labor		$2.10
Teaming sand and cement		0.39
Brick		[illegible]
Cement		[illegible]
Sand		[illegible]
		[illegible]

Total cost per lineal foot, $1[illegible].17.

Cost percentages: For materials and plant, 53 per cent. For labor, 47 per cent.

The piling and timbering of the tracks was done by the railroad company at their own expense. This hand excavation cost $1.25 per cu. yd.

In another case the steam shovel could not take out the bottom on account of the proximity of a viaduct. This earth was scaffolded out at a cost of $1.06 per cu. yd., being handled four times before it reached the spoil bank.

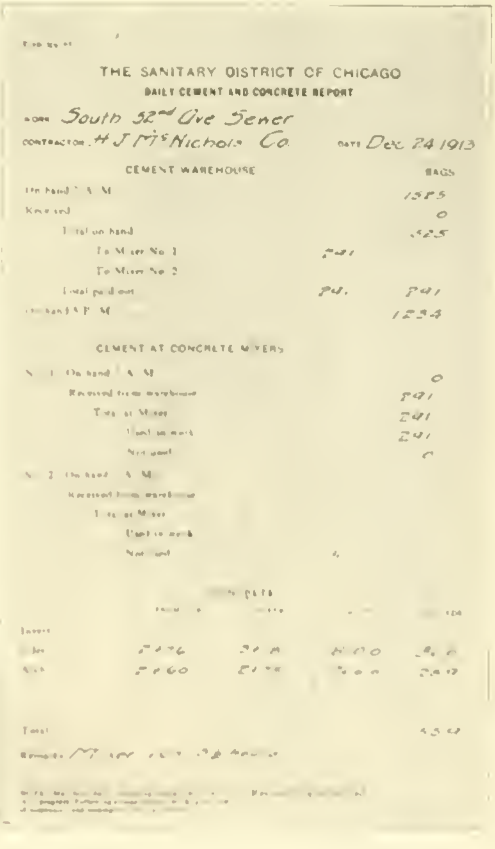
The moving of the steam shovel a distance of 1,050 ft. across a railroad yard and over the tunnel section was $560, or 53 cts. per foot. This includes the partial dismantling of the shovel to pass under obstructions. At the start the shovel was taken off the railroad spur, moved ½ mile and placed on timbers to span the trench, at a cost of $750.

A special section was designed near the outlet at Ogden ditch, where the ground surface was low and the bottom very soft, on the assumption of no cut and no cover, with the sewer flowing full. A foundation of a double layer of 2-in. plank was laid before the brick work was started.

TUNNEL SECTION.

The tunnel section, 1,050 ft. long, extends under the Morton Park yard of the Chicago, Burlington & Quincy Railway, and passes directly under five piers of the viaduct carrying South 52nd Ave. over the railroad yard. In places there was only 12 ft. of covering over the roof of the tunnel. The ground was stiff blue clay, containing but one sand pocket, which caused some earth settlement, visible at the ground surface. There were no settlements whatever at the piers. The unit costs for the tunnel work are given in Table VII, and the gang organizations in Table VIII.

The work was carried on by two night shifts of miners and muckers and one day shift of bricklayers, working 8 hours each, or a total of 24 hours per day. One shaft was sunk, from which two headings were run. In Fig. 3 is shown the method of timbering in good stiff clay. In poor ground, the crutches would be made longer, with the



THE SANITARY DISTRICT OF CHICAGO
DAILY CEMENT AND CONCRETE REPORT

WORK South 52nd Ave Sewer
CONTRACTOR H J McNichols Co. DATE Dec 24 1913

CEMENT WAREHOUSE

		BAGS
On hand 7 A. M.		1525
Received		0
Total on hand		1525
To Mixer No. 1	291	
To Mixer No. 2		
Total paid out	291	291
On hand 5 P. M.		1234

CEMENT AT CONCRETE MIXERS

No. 1 On hand 7 A. M.	0
Received from warehouse	291
Total at Mixer	291
Used in work	291
Not used	0
No. 2 On hand 7 A. M.	
Received from warehouse	
Total at Mixer	
Used in work	
Not used	

Fig. 9. Specimen Daily Report Form for Reporting Use of Cement and Concrete on Sewer Work, Sanitary District of Chicago.

lower end set below the spring line, and the 2x10-in. plank at the roof would be placed closer together. The excavated material was dumped from the shaft into railroad cars and hauled 3 miles to Western Ave.

The method of setting up the centers for the arch after the invert is built and timbering removed is shown in Fig. 4. The loose brick seen inside of the invert support the centers, spaced 4 ft. apart; 2x4-in. lagging is then placed. The earth at the roof is supported by 2x4-in. props resting on the lagging, reinforced by a 2½-in. iron prop extending from the floor of the invert to the crown plank at roof.

The average progress for 24 hours was 12½ ft. in each heading, or 25 ft. per day for both headings. The average number of brick laid per 8 hours per bricklayer was 3,000.

SOUTH 52ND AVE. SEWER OUTLET—SANITARY DISTRICT SECTION.

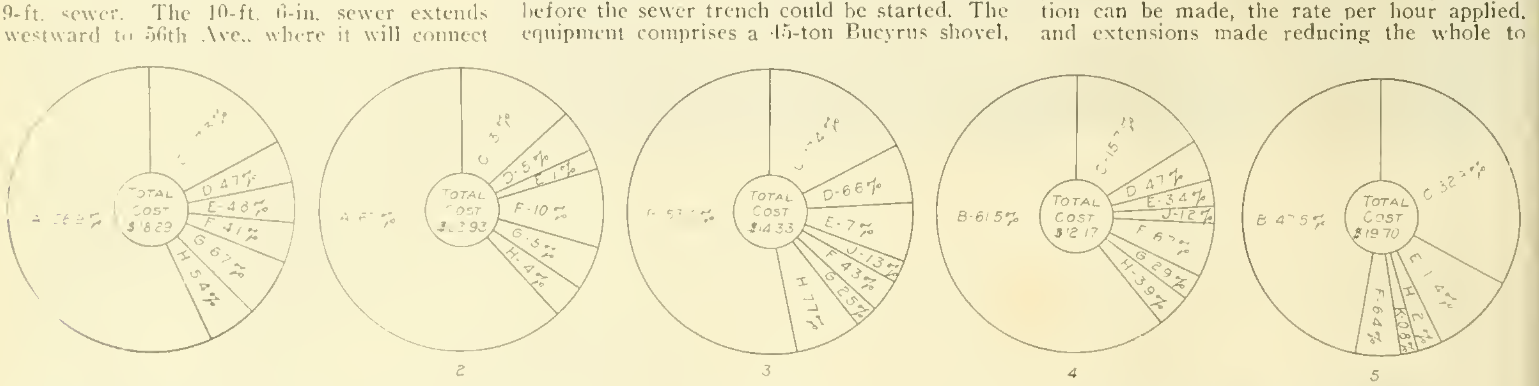


Fig. 10. Diagram Showing the Cost per Lineal Foot of Large Concrete and Brick Sewers with Proportional Distribution of Field Costs.

(1)—12x14 ft. plain concrete section of W. 39th St. conduit, avg. cut, 23 ft. 6 ins. (2)—12x14 ft. reinforced concrete section of W. 39th St. conduit, avg. cut, 22 ft. 0 ins. (3)—7 ft. brick, Cicero section of So. 52nd Ave. sewer, avg. cut, 21 ft. (4)—7-ft. 6-in. brick, Cicero section, of So. 52nd Ave. sewer, avg. cut, 20 ft. (5)—7-ft. 6-in. tunnel, Cicero section, of So. 52nd Ave. sewer.

Legend: A = Concrete masonry; B = Brick masonry; C = Excavation; D = Backfill; E = Waste disposal; F = Miscellaneous; G = Coal; H = Lumber, J = Pumping, K = Power.

This work is now under contract. The main section, as shown in Fig. 5, consists of a 12x16-ft. horseshoe shaped sewer, the outfall being at the main channel of the Sanitary District at South 52nd Ave. This section extends north 1,800 ft. from the canal to a point where it will divide into a 10-ft. 6-in. and a 9-ft. sewer. The 10-ft. 6-in. sewer extends westward to 56th Ave., where it will connect with the 8-ft. 6-in. Oak Park sewer and the Berwyn 5-ft. sewer, both of which now discharge into Ogden ditch. The 9-ft. sewer will extend north, across the Ogden ditch, and connect with the 7-ft. 6-in. Cicero sewer above described. This section has a grade of 1 ft. in 5,000 ft., the grade being largely fixed by the elevation of the outlets of existing sewers. The dividing wall, a detail of which is shown in Fig. 6, was designed to keep high velocities in dry weather and avoid deposits, the intention being to keep the dry weather flow on one side of the wall. At the upper end of the 16-ft. sewer, stop planks are arranged to divert the flow to either side of the dividing wall.

TABLE VI.—CONSTRUCTION FORCE AND RATES OF PAYMENT ON CICERO SECTION OF THE SOUTH 52ND AVENUE 7-FT. AND 7 FT. 6-IN. BRICK SEWERS, TO ACCOMPANY COSTS IN TABLES IV AND V.

Employes.	Wage per day.
1 superintendent	$8.00
2 foremen, each	6.00
1 shovel engineer	8.00
1 hoisting engineer	5.60
1 cranesman	4.70
1 shovel foreman	3.25
1 derrick foreman	2.75
2 pump foremen, each	3.00
1 watchman	3.00
1 bricklayer	12.00
5 bricklayers, each	10.00
3 tenders, each	3.75
4 cement mixers, each	3.00
5 cement carriers, each	3.25
4 to 8 bottom men, each	3.75
5 bracers, each	4.10
2 center men, each	3.75
1 blacksmith	3.50
1 blacksmith helper	3.00
[illegible] scaffold men, each	2.75
3 brick tossers, each	2.25
4 brick wheelers, each	3.00
6 roller men, each	2.80
1 material man	3.00
1 timekeeper	3.00
2 waterboys, each	1.00
10 to [illegible] common laborers, each	2.00
1 to [illegible] teams, each	6.00

The contractor began work at the outfall and has now completed a portion of the 16-ft. section. It was necessary to excavate through the spoil-bank of the Main Drainage Channel before the sewer trench could be started. The equipment comprises a 45-ton Bucyrus shovel, with 1¼-cu. yd. dipper, mounted on 36-ft. timbers to span the trench. The average cut is now 17 ft., with a width of trench of 21 ft. 10 ins. Excavated material is loaded from the shovel into 4-cu. yd. dump cars and deposited in backfill, and the excess is hauled about ¼ mile to the spoil area provided by the Sanitary District.

TABLE VII.—UNIT COSTS OF CONSTRUCTING THE CICERO SECTION OF THE SOUTH 52ND AVENUE 7-FT. 6-IN. BRICK SEWER IN TUNNEL.

Item.	Cost Per lin. ft.	Cost Per cu. yd.
Excavation	$6.49	$2.43
Waste disposal	2.05	0.77
Lumber	0.40	....
Electric power	0.15	....
Miscellaneous	1.26	....
Brick masonry	9.35	....
Labor		$4.40
Brick		4.09
Cement		0.57
Sand		0.29
		$9.35*

Total cost per lineal foot, $19.70.

Cost percentages: For materials and plant, 27 per cent. For labor, 73 per cent.

*Note—Masonry runs 1 cu. yd. per lineal foot of sewer.

The entire work is to be built of concrete, of a mixture approximately 1:2½:5, except for the dividing wall, which is approximately 1:2:4. Stone is used for the coarse aggregate. Pouring of the concrete consists of three operations; the invert, sides, and arch. Forms for the invert can be seen very plainly in Fig. 7. The invert and dividing wall in the 16-ft. section are reinforced with steel throughout, and the arch for about 100 ft. where the sewer passes under the Chicago & Northwestern Railway. The reinforcing is placed in invert to take care of any unequal settlement due to soft ground.

Below the springing line the side forms are built of 1-in. flooring, nailed to 6x6-in. uprights, 4 ft. on centers, braced by a 6x6-in. cross timber at the springing line, supported by a 6x6-in. post near the center of the sewer, with diagonals extending to the vertical posts. Above the springing line, 6-in. 8-lb. channels are used, curved to the proper radius, and reinforced with a 4x4-in. angle, 8 ft. long at the quarters. On these ribs spaced 4 ft. on centers, 2x6-in. lagging is placed about 6 ins. apart, on which sheets of No. 18 gage sheet steel is laid, lapping about 6 ins. For the dividing wall the form is built up of wooden sheets 8 ft. long, suitably braced against the side walls, with steel separators between to maintain the proper spacing.

The contract prices for the work are on unit quantities of excavation, concrete, reinforcing steel, etc. These units assembled on the different sizes of sewer with approximate quantities, make a contract price per lineal foot about as follows:

16-ft. section in 17-ft. cut	$40.63
10½-ft. section in 12-ft. cut	15.48
9-ft. section in 14-ft. cut	12.83

COSTS AND COST KEEPING METHODS.

Cost data, to be of any value, must be founded on a correct daily report on labor, a weekly or a monthly report on material. About half an hour spent on this work each day should prove ample on sewer work to obtain the needed information.

The contract prices for material can usually be obtained from the contractor, and one sheet only made out for this, applying the unit prices to the monthly statement.

For the use of the Sanitary Division of the District, a daily report was devised for labor and progress of the work, designed to fit either a brick or concrete job. Figure 8 shows such a form as filled in for one day. The distribution of hours is made at the time the form is filled in. The rate per hour need not be filled in daily, since this can be covered on one sheet showing the rates for the entire job. From these reports, a monthly tabulation can be made, the rate per hour applied, and extensions made reducing the whole to a money basis. At the end of the job, the monthly statements can be compiled, making

TABLE VIII.—CONSTRUCTION FORCE AND RATES OF PAYMENT ON 7-FT. 6-IN. BRICK SEWER IN TUNNEL, TO ACCOMPANY COSTS IN TABLE VII.

Note.—The rates are for 8-hour shifts, and each force is for two headings, one-half the force working per shift being in each heading.

(a) First Shift—8 a. m. to 4 p. m.—Bricklaying

Employee.	Wage per day
1 superintendent	$10.00
4 bricklayers, each	10.00
6 tenders, each	3.75
2 assistant tenders, each	3.25
2 cement mixers, each	3.25
2 car pushers, each	2.50
2 shaft tenders, each	2.50
1 hoisting engineer	5.60

(b) Second Shift—4 p. m. to 12 midnight—Mining.

Employee.	Wage per day.
1 foreman	$6.00
6 miners, each	3.75
4 muckers, each	3.60
2 car pushers, each	2.50
1 shaft tender	2.75
4 laborers, each	2.50
1 hoisting engineer	5.60
1 timekeeper	3.00

(c) Third Shift—12 midnight to 8 a. m.—Mining

Employee.	Wage per day.
1 superintendent	$10.00
8 miners, each	3.75
6 muckers, each	3.00
2 car pushers, each	2.50
1 shaft tender	2.75
4 laborers, each	2.50
1 hoisting engineer	5.60
1 timekeeper	3.00

(d) Miscellaneous.

Employee.	Wage per day.
1 dump foreman	$2.50
6 to 12 laborers, each	2.00
1 blacksmith	3.50
1 electrician	4.00
1 carpenter	4.50
1 carpenter's helper	3.00

The average progress per day was 12½ ft. per heading or a total of 25 ft. per day.

one tabulation for the entire job, and the weekly reports for material can be handled in the same way.

A daily report for material was also devised, showing the amount of cement in the warehouse, the amount used in the work, and the cubic yards of concrete placed. Figure 9 shows such a form as filled in for one day. The tally on cement can be kept by the inspector at the mixer as the bags are delivered. The inspector can also fill in the station numbers showing progress. The cubic yards placed can be entered by the engineer in charge. A small table showing the cubic yards per running foot of invert, sides and arch, is a very handy thing to have so that quantities can be entered without any extended computation.

The tables on cost data here shown are intended to cover all field operations, including superintendence, labor, material and plant. Overhead charges of the contractor are not included, such as office expenses, bonding, liability, insurance, and discount on municipal bonds on special assessment work.

The item of plant charge is a difficult one. For instance, take the item of steam shovels. There are steam shovels in service today that are 25 years old, whereas others are worth only scrap value at the end of three or four years. Many contractors charge off the entire plant to the job. So far as I see, for machinery such as steam shovels, dinkies, etc., it is fair to spread the plant cost over a period of ten years, allowing interest at 6 per cent on first cost, thus making a depreciation charge of 16 per cent per year. Alterations, fitting up, freight, small tools, etc., are directly chargeable against the job, and should be added to the above 16 per cent on cost of machinery and similar equipment for total plant charges against any piece of work.

In considering the overhead charge to be made, some figure must be taken in making up an estimate. This is more apt to be too small than too large. In Illinois, liability insurance will cost from 7½ to 11 per cent of the payroll, and on work described in this paper, the labor item is about 50 per cent of the total field cost for open cut work, and about 70 per cent for tunnel work. This makes a charge of 3½ to 8 per cent of total for insurance, to begin with. Office rent, telephone, cost of getting work, and other items may increase this to 10 or 15 per cent. Adding 15 per cent for profit, we thus have 25 to 30 per cent to add to the field cost. Figure 10 shows the cost per lineal foot on the various jobs on which cost data are here given in the tables and also shows the field cost percentages for the component parts of the various jobs. With the aid of the data contained in the tables, reinforced by current market quotations on material, the author has made estimates for similar kinds of sewer work, in the aggregate about one-half million dollars. Such estimates agree with the low bid within 4½ to 7 per cent. In applying any data, a word of caution is necessary, since two jobs are seldom alike. In transferring the unit costs for work already performed to new estimates, due consideration must be made for differences in the local conditions, character of the soil, increased cost of labor, and the availability of standard types of machine to handle the work.

The contractors on the various jobs mentioned herein were as follows: On the West 39th St. conduit, John W. Farley & Co. built both the Sanitary District and city sections. On the 52nd Ave. job in Cicero, James Healy was the contractor. On the 52nd Ave. job now under construction for the Sanitary District, the H. J. McNichols Co. are the contractors. To them is due acknowledgment for courtesies of information during the progress of the work.

The construction work described herein was carried on and the data were compiled by the writer as resident engineer for the Sanitary District of Chicago, under the direction of Mr. Langdon Pearse, Division Engineer, and Mr. George M. Wisner, Chief Engineer. Mr. S. A. Greeley, Assistant Engineer, has been in charge of the design on the South 52nd Ave. sewer.

General

The Functions of the Engineering Society.

The recent address by Mr. L. K. Sherman, president of the Illinois Society of Engineers and Surveyors, at the annual meeting of the society, calls attention to the fact that engineers and engineering bodies are giving increased attention to matters affecting their welfare. This address in full is herewith given:

The constitution of almost all engineering societies state that "the objects of the organization are the exchange of technical knowledge and experience and the advancement of the engineering profession." The benefits derived from the presentation of papers and the exchange of professional experience are of themselves ample reason for membership in the society, and incidentally they are large factors toward the professional advancement of engineering.

In my opinion, however, the time is now ripe when the engineering societies should enlarge their field of endeavor from the purely technical and literary functions to engage directly in efforts promoting the material status of the engineering profession and the individual members thereof.

I am not advocating this effort to promote the material status of engineers because of any alarming conditions. Engineering has advanced from a trade to a profession. Figures gathered by engineering societies, records of the graduates of technical schools and pay rolls of corporations show that the average annual income of the civil engineer today compares moderately well with similar statistics of the lawyer and doctor.

The field of the engineer has broadened, especially in lines of sanitation, highway construction and reclamation of lands. His position before the public is better recognized. Engineers are being placed as town managers, on public commissions, as investigators for financial corporations, and as constructors, instead of letting the work by contract.

It is, however, not our place merely to accept what the conditions of the times offer; it is our duty to take advantage of and to push those conditions which result in our rightful material welfare. This can be accomplished only through organization. This is the day of organization—individual effort fails. In business, consolidations alone survive. If engineers are to progress and to retain their rightful sphere they must do so through organization. The legal and medical professions are strongly organized to guard and advance their interests in legislation and in public relations. The architects of this state have an able organization, which legislated a certain kind of structural work out of our hands into theirs. Almost all lines of business have their lobbies. However much we may disagree with the methods of the building trades union we have to admire the efficiency of such organization. Some factors that have contributed toward the delinquency of engineers furthering their material welfare are their modesty, the fact that they are migratory animals, their false sense of dignity and too much conservatism. One of the unavoidable handicaps of the civil engineer arises from the very nature of his work. His work ends when the particular structure which he may be engaged on is completed. He then must seek other and often remote fields where new construction is to be started. These intervals between jobs are often of frequent occurrence. Thus the engineer is a migratory animal. I do not mean to underrate the desirability of maintaining the dignity of engineering as a profession, but dignity should not interfere with business progress with us any more than it does with law or medicine.

I once heard of a secretary of an engineering society who was so conservative that he did not believe in any co-operation with the societies and in fact scarcely believed in a meeting of his own society.

The Illinois Society of Engineers and Surveyors has been one of the few engineering associations to enter the field of promoting material professional welfare. The society originated a movement to demand justice for a member engaged on a government bureau at Washington. Our action and its vindication were highly commended by the technical press, but the conservative [illegible] of many engineering societies prohibits this sort of work. During the past year a proposition was submitted at Springfield to place the State Geological Survey under the Board of the State University. This matter was brought to our attention and after consideration by the trustees and officers of the society it was decided that the proposed change would be detrimental to the efficiency of the Geological Survey, and communications were sent to a number of legislators at Springfield to that effect. The change was not made and the Survey maintains its separate existence.

After the passage of the law creating a State Public Utilities Commission, this society presented a petition to the Governor urging the appointment of an engineer upon said commission. Our membership is now honored with a member of the State Public Utilities Commission. In a number of instances the society has had special committees working in the interests of legislation affecting surveys and matters pertaining to land drainage and special assessment work.

The technical press of the country during the past year contains more communications than in any previous period on "The Status of the Engineer"; "Compensation of Engineers"; "Engineers' License Laws"; "Code of Professional Ethics"; and "The Consolidation of the Engineer's Work in Large Engineering Corporations," etc. An association of consulting engineers in New York has been actually formed to handle these matters. The formation in Chicago of a Technical League, a union of engineers associated with the American Federation of Labor, is a recent and most radical development. All of this is an indication that the engineering society is not entirely meeting the problems of material professional welfare.

Among the plans, projects and suggestions presented for advancing the status of engineers are: The licensing of engineers and surveyors; civil service laws; legal compensation for city engineers, etc. [illegible]; code of professional ethics; standardization of fees; and employment bureau operated by the societies.

I cannot attempt here to consider the [illegible] arguments for and against these plans. What I am advocating here is that it is the proper duty and function of the engineering society to consider such plans and to have an organization which possesses the necessary executive machinery to act upon those measures which are of benefit and to circumvent those which are detrimental to the engineer and his allied work.

I recommend that the trustees and officers of this society be authorized to act as an executive committee in the name of the society in all matters of legislation and public relationship, and to receive, consider and act upon, or report upon, any complaint or suggestion presented by any member of the society. I also advocate the appointment of a committee on legislation whose duties it is to follow and to report to the executive committee any proposed state or municipal legislation which might be of interest.

The local or state engineering society is the logical organization to initiate such measures as are conducive to the material betterment of engineers. The efforts initiated by the local society should in turn be supported by the action of the interstate or national engineering societies.

The ability to accomplish practical results will to a large extent depend upon the numerical strength of the society. The voice of the society should be the representative voice of all engineers in the state. I believe that if the state society would acquire the reputation for action in measures affecting the status of the engineering profession and especially of its individual members there would be an immediate and large addition to its membership list.

In closing, to those who may think that all these efforts are impracticable of accomplishment, I would say:

"Heaven is not reached
At a single bound.
We must mount the ladder
By which we rise
Round by round."

Water Under a Dam—the Stony River Failure.

Sirs: The failure of the dam at Stony River in West Virginia demonstrates again what has been demonstrated scores of times, namely, that it is inviting disaster to design a dam without considering the effect of water under it.

This failure will no doubt be set down by the investigators in the standard way—poor foundation, mistake in judgment, unforeseen contingencies.

Here is something for the investigators to ponder, and for all engineers to consider: *No dam ever failed that was heavy enough and wide enough at the base to be stable under the pressure of water on the wetted side and underneath it, regardless of what it was founded on.*

I have shown a number of times, and anyone may work it out graphically in a very simple manner, that the resultant of the pressure on the wetted side, the pressure of the water beneath the dam, and the weight of a honey-comb dam passes nowhere near the middle third, in fact may be outside of the base entirely—may even be *upward*. This one has a very large upward angle. What stability can there possibly be under these conditions?

That there was water beneath the horizontal slab of this dam is absolutely proven. It was coming out of the weep holes before the failure. That this water was expected to be there is shown by the very fact that weep holes were provided. The presence of the weep holes is also evidence that the dam was not built to withstand the upward pressure of the water under the slab, or provision would not have been made for it to run away and waste.

To go by easy stages, what does water do in winter? It freezes, of course. What is to prevent the water gently oozing out of these weep holes to freeze in the hole or in the ground immediately surrounding them and form plugs? Would not the ground on the lower side of the dam also freeze? Now, what condition have we? Water beneath the dam attaining its full pressure because it has no means of escape; a dam that is far from stable against this pressure; an unresisted tendency for the dam to rise from its foundation and break up or be pushed down stream. A small rising of the dam would admit an influx of water which would wash out the ice plugs. The mud and water below the bottom slab of the dam would then begin to flow out through the weep holes. More water would enter through the cracked cut-off wall. In a short time enough mud would be washed through to admit a stream of water. This would continuously increase in volume until the dam was sufficiently underwashed to cause its partial collapse.

The failure of this dam is just as logical and natural as anything could possibly be, and it is due to the design as clearly as anything could be.

This is not the first time a hollow dam like this one has been underwashed and has failed. Other lightweight dams have also failed when the under pressure lifted the structure and opened a way for water to flow out and scour a way for more water.

One dam constructed like this one, with its bottom slab and its weep holes and *two cut-off walls*, was underwashed to such an extent that there was room for wagons to drive under it. The dam happened to hold together showing how very light it was. The dam referred to was at Pittsfield, Mass., and the failure occurred in *midwinter*, namely, Jan. 7, 1909. The experts reported that the cause of the accident was "entirely with the foundation." *Mother Earth* blamed again: the same old primitive idea of the three elements causing all of man's trouble. In this case water and earth combined to do the damage. Man, of course, had nothing to do with it. He just built a dam the cheapest way possible; otherwise he would not be called an engineer. Anyone can spend a client's money. It takes an engineer to save it. Anyone can see that water under a dam will exert a pressure to lift it. It takes an engineer to construct specious arguments that this water is simply present by capillarity and is totally immune to any other laws of nature while lying beneath a dam.

Let us look at this foundation failure business. Did these foundations fail because of too great a pressure from the overlying structure? If so, why did the structure hold together and let the foundation run away from under it? Why did one structure hang in the air after the failure, if it was so heavy?

If the foundation did not fail from great pressure, what was the nature of the failure of the foundation? Earth dams exist without any masonry—dams made merely of mud. Imagine a dam of this sort to be transformed by converting the crest triangle into a hollow shell with very little weight. Water gets under this shell and lifts it enough to make a horizontal fissure through which a stream quickly starts, carrying mud with it. Mud is scoured away under this shell until the whole is underwashed and either remains suspended or breaks up and falls in chunks into the hole. Is this a failure of the foundation? If so, why was the foundation good in the case of the earth dam and not in case of the dam with the hollow shell at the crest? In the latter case the foundation was subject to very much less pressure. This is exactly the case of a honey-comb dam or any lightweight masonry dam. All of the scores of foundations which have failed have been too light in weight to stick in the mud against the uplifting pressure.

Of course the foundation will be disturbed, millions or billions of gallons of water washing through a foundation would naturally leave some trace. A charge of dynamite, set off under a structure, would disturb the foundation but no one would say that the damage was due in this case to the insufficiency of the foundation.

Provision should be made for the pressure under a dam for exactly the same reason that the pressure on the up-stream side under the fillet of mud near the base must be provided for. *This pressure was not provided for in the Stony River Dam.* Weep holes and drainage are absurd in a climate where water freezes.

It is a crime to design a dam without considering upward pressure.

Very truly yours,
Edward Godfrey.

Monongahela Bank Bldg., Pittsburgh, Pa.
Feb. 5, 1914.

Some Notes on Costs of Loading Motor Trucks From Ground Storage Piles.

Contributed by H. D. Pratt, Engineer, Link-Belt Co., Philadelphia, Pa.

The ability of the motor truck to carry bulk material long distances in quantities and at a high rate of speed makes it possible materially to reduce the cost of transporting crushed stone, coal, sand, gravel, etc. The speed of travel is high as compared with the horse-drawn vehicle. The unloading is automatic, the truck body being dumped by lever by the operator. In many cases the truck is loaded from an overhead chute, but more often the material to be moved lies on the ground in storage piles, not only in regular storage yards, but in streets, roads or wherever most convenient. The high cost of loading motor trucks with material in ground storage is one of the factors which have retarded the sale of trucks for this work.

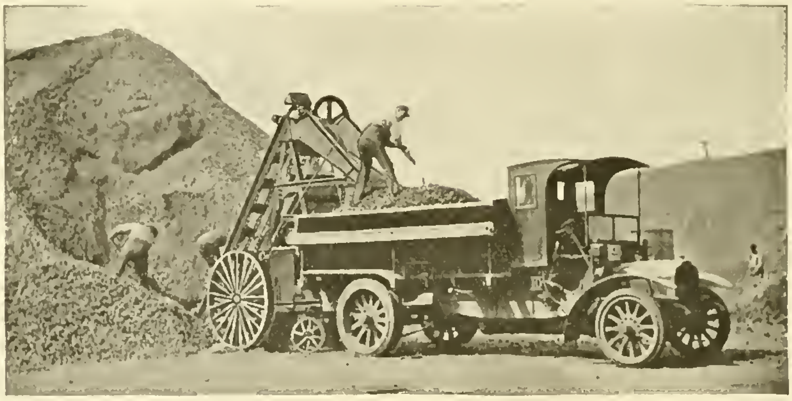


Fig. 1. Loading Crushed Stone Into Motor Truck With Portable Truck Loader.

The truck must necessarily be high to hold the load. Loading by shoveling means that the truck spends more than half its time waiting at the storage pile to be loaded, and between trucks the shoveling gang is idle. A good average day's work for a shoveler is 20 tons of gravel, less of stone, and slightly more of sand. Not only is the truck standing idle while being loaded, but the loading by hand costs 8 to 12 cts. a ton.

The portable truck loader shown by Fig. 1 will materially reduce these loading costs. As shown a bucket elevator is mounted on wheels; with power supplied by a motor or gasoline engine, this elevator will load sand, gravel or coal at the rate of one ton per minute. The truck driver trims the truck with a shovel, and one, two or three men, depending on the nature of the material to be handled, trim and feed to the loader. This combination will load the truck at a cost of 2½ cts. to 5 cts. per ton, the higher cost being that of handling crushed stone, with coal at the low end, and sand and gravel between. The saving over hand loading is thus about 6 cts. per ton.

There is also a growing demand for a further combination of truck and loader, namely, a truck with loader elevator mounted on the rear end of the body, arranged to lower into a storage pile when the truck is backed up to it, and also to raise clear of the ground when not in use. The elevator is supported independently of the dumping body, and is driven through a clutch by a connection to the truck transmission.

The loaders described above are made by the Link-Belt Co., Philadelphia and Chicago.

Engineering and Contracting

Devoted to the Economics of Civil Engineering Design
and to Methods and Cost of Construction

Volume XLI. CHICAGO, ILL., FEBRUARY 18, 1914. Number 7.

Efficiency at the Source of Management.

'Efficiency" and "scientific management" are terms which occupy a very important place in the vocabulary of engineers and managers. We have been told, especially in factory management, how production can be enlarged by increasing the efficiency of the workmen by lessening the number of motions required to perform a given task, and by eliminating waste, thus making it possible for a man, by working with machine-like precision, greatly to increase his output. The prominence of the term "efficiency" and the wonderful claims of "scientific management" have made them tools fashioned in such a shape as best to fit the hands of those at the top of organizations. Attention has been centered on the fact that workmen are capable of greater production. In many cases these workmen have been given actual money for increased efficiency; while in some cases they have not profited as much as have those for whom they worked. All of this increased efficiency has one practical effect—it places more money in the hands of men at the head of organizations. Those who have invested funds in these companies certainly have the right to expect just as great efficiency on the part of the officials who represent them.

Possibly the above terms will not be used with such frequency by those at the top if there prevails the somewhat radical ideas of men like Roger W. Babson, who, in addressing a conference of the National Efficiency Society, stated that the investigators and engineers employed to improve business methods would, if honest and courageous, generally advise "the firing of the president, the treasurer, and most of the directors." As expressed by Mr. Babson, the greatest inefficiency is often at the top, for many directors attend meetings only for fees; they know little or nothing, and care less, about the business. Other directors and high officials hold their positions because of pull or inherited property or family connections, and have no fitness for their work. Finally, he said, the employes largely base their own objections to ratings and markings under the "scientific" scheme on the fact that many of their superiors are incompetent, indifferent and overpaid. Many men—especially those at the top—will consider the above statements extremely radical, but when they are considered seriously is there not much truth in them? We must form our conceptions of efficiency by comparison, and we usually look toward the top of the pay roll—not at the bottom for our standard of comparison. Of course there is some danger in taking this viewpoint too seriously, because the man below is apt to find too much consolation in the thought, and possibly may underrate the ability and efficiency of his "boss" on account of a misunderstanding of the conditions under which the latter works. The thought suggested by Mr. Babson is not entirely new, as the ability of those in direct charge of organizations is often questioned in the minds of workmen, even if it does not find expression. It can not be doubted that more efficient methods will greatly improve the effectiveness of labor, but the possession, or lack, of efficiency at the top of the organization certainly has a great effect upon the return to the investor.

In the effort put forth by those engaged in scientific management and appraisal work attention may well be centered first upon the heads of organizations instead of starting at the bottom and working upward, often stopping at the half-way point. The same thought applied to municipal affairs, to railroad corporations and to engineering firms should produce beneficial results. If this policy is adopted much non-essential discussion and cant will be arrested at the source. We realize that one fact many serve to hinder progress, namely, that those who vote funds for efficiency investigations are usually at the top. However, this need not deter those engaged in work authorized and conducted by the government.

The Consulting Engineer.

An article of exceptionally wide appeal to the interest of the engineer readers of this journal is published in this issue since it is probably an early aim of nearly every engineer, and one which is renounced only with some regret as years go by, to become a consulting engineer. The article to which we refer is entitled: The Consulting Engineer in Water Works Practice. The title given the article is somewhat restrictive since the application of the general statements made relative to the training, experience and qualifications necessary to artistic and financial success as a consulting engineer, are later applied to the special case of the consultant in the water works field.

This article is the first of a series of special contributions to the Water Works Section of this journal which will be published during the present year. The contributors of these articles are Messrs. J. N. Chester, George W. Fuller, John H. Gregory, Dow R. Gwinn, Allen Hazen, James Nisbet Hazlehurst, George A. Johnson, Wynkoop Kiersted, Emil Kuichling, Dabney H. Maury, Leonard Metcalf and R. O. Wynne-Roberts.

To return to the article mentioned, it is not our purpose here to present its salient features, points and conclusions. We believe that all of our engineer readers will find this article of great interest in its general discussion of the life and work of the consulting engineer. In its special application to water supply engineering the article can be read with profit by all those to whom is entrusted the management of local water supply systems. There are several ways in which the consultant can be of special service to the local management and these ways are enumerated and discussed.

Additional Comment on the Description of Their Failures by Designers.

Under the caption, "My Greatest Mistake," *The Contract Record* of Toronto recently commented editorially upon our editorial of recent date entitled Should Engineers Describe Their Failures as Well as Their Successes? Our contemporary appears to consider our suggestion not only impractical but somewhat amusing. In writing the editorial to which we have referred we considered it a certainty that some readers would take the view of the matter held by our contemporary.

It will be recalled that our editorial stated that two prominent consulting engineers agreed, in a recent conversation, that a series of letters published under the general caption, "My Greatest Mistake in Engineering Practice," would be of great value to the profession. We doubted the practicability of securing for publication such a series of letters but commended the plan on the modified basis that it is incumbent upon a designer who has published a description of one of his designs to make public later the nature of any considerable shortcoming revealed in that design after it had been placed in use. This was the point we desired to emphasize and we do not think our contemporary adhered to the text very closely in its good natured comment here quoted:

> To our mind this provides a rich and rare exemplification of what is all right in theory but poor in practice. Imagine the reminiscences of a score of our leading Canadian municipal engineers published in book form under such a heading as that suggested! In the first article City Engineer Brown would tell how he made a mess of the water supply at Hogtown, and in the second City Engineer Jones would narrate how he wasted thousands of dollars in under-estimating the requirements of the city's sewerage department. Records of failure written in a general way are well enough handed down from the last generation, but we live in an age when nothing counts like success. However much we might profit by the mistakes of others, to advertise our shortcomings as designers and constructors would be inviting the fate of the dog who after being given a bad name came to the ignominious end of hanging. There may be one or two outstanding geniuses in the engineering field whose records would survive any publicity given to their greatest mistakes, but it would spell *finis* to the professional career of the rank and file in the engineering community.

While the comment above quoted appears to be directed rather to the broad suggestion of the engineers we mentioned in introducing the subject than to our rather narrowly limited and much more conservative suggestion we quote it as an interesting statement of the other extreme view on the general subject. In our previous editorial we made no effort to exhaust the subject nor shall we do so at this time, but perhaps it would be well to state some of the points we previously took for granted.

To illustrate the character of failures which we had in mind let us assume that an engineer designs a concrete dam along well recognized lines for the greater part but differing somewhat from other dams in some important detail, for example, in the design of the flood gates. Let us suppose, also, that the design of the gate adopted seems to give promise of doing good service and of possessing certain advantages over older types. Again, suppose that the designer describes his design in an article contributed to some technical journal and calls particular attention to the flood gate by illustrations and descriptive matter. Now our point was that should the gate prove deficient when put to the test the designer should give the same publicity to that fact that he gave to the first statement of what he believed his design would accomplish. We cannot think that our suggestion is impracticable or that its adoption would injure the reputation of the designer among thinking men who realize that mistakes of this nature are being made frequently and by everybody who does much work.

Our contemporary uses in an illustration of its view of the broader aspect of the description of engineering failures a reference to making a "mess" of the engineering for a city's water supply. A failure which can be characterized as a "mess" is usually described by others and unless the designer desires especially to state his side of the case there is little left for him to say. Moreover "messes" of engineering work need no longer be made by those who will take the trouble to become well grounded in such fundamentals as the text books set forth. If an engineer makes a failure of a system of pipe sewers it can have but slight interest to the profession as a whole, for thousands of engineers are now entirely competent to carry such works to successful completion. But if an engineer tries out a new type of sewage tank and it fails his reputation will not suffer from a recital of his apparently sound reason for adopting the design and a statement of the tank's failure to accomplish the results desired.

Roads and Streets

Cost of Constructing a Concrete Pavement Surfaced With Asphaltic Oil and Stone Screenings for Huntington Drive, Alhambra, California.

Contributed by Earnest F. Ayres, Corvallis, Oregon.

The question of whether a surface pad of asphaltic oil and stone screenings will stick to a concrete pavement base or peel off in hot weather is of paramount importance in California since the State Highway Commission and many of the counties adopted the concrete road as a standard for all construction under the bonding acts. No one doubts the permanence of the Portland cement concrete, but its lack of resiliency, its liability to crack under temperature and atmospheric changes, and the difficulty of repairing it cheaply and satisfactorily makes it necessary to provide some form of surfacing. If a thick asphaltic concrete is specified the cost of the improvement is excessive; if a thin surfacing is used the cost is small, but—will it stick?

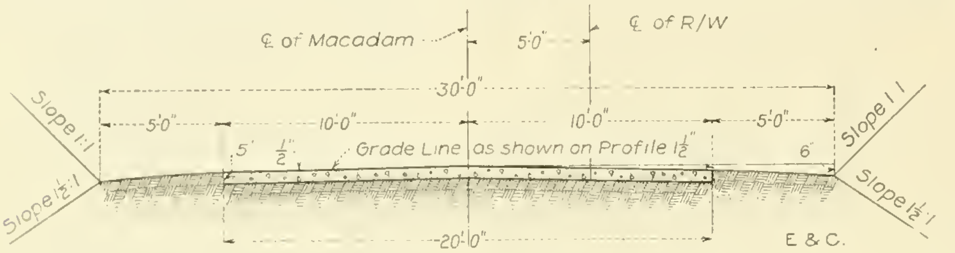


Fig. 1. Cross-Section of Concrete Pavement Surfaced With Asphaltic Oil and Stone Screenings, Huntington Drive, Alhambra, California.

Frank H. Joyner, chief engineer of the Los Angeles Highway Commission, thinks it will, and specified this type of pavement for the Huntington Drive. This road consists of a 5-in. Portland cement concrete base, surfaced with ¼-gal. of asphaltic oil and 0.0063 tons stone screenings to the square yard. Figure 1 shows the cross-section of the pavement. The base of the pavement is shaped parallel to the finished surface, the concrete being the same thickness through the cross-section. The shoulders were formed of the material found in the roadway, and the ditches are patterned after the Massachusetts standard, 6 ins. deep, and of a width which can be called anything up to one-half the width of the finished road. The oil was applied at a temperature of 180° to 250° F. by a 1,000-gal. cylindrical tank mounted on a 5-ton Peerless motor truck. The distributing tank is shown by Fig. 2. The screenings were spread while the oil was still hot as shown by Fig. 3. The earth base was plowed with a Petrolithic gang rooter preparatory to tamping which was done with a Petrolithic rolling tamper. A view of the gang rooter at work is shown by Fig. 4 and Fig. 5 shows a 3-wheel road roller finishing a section of sub-grade ready for concrete laying.

The following tabulation shows the amount of the different mixtures used in the concrete base for the pavement:

Mixture—	Sq. yds.
1:2:4	19,096.4
1:2.75:4	568.9
1:2:4.27	621.4
1:2:4.5	1,324.3
1:2:4.6	711.1
1:2:4.67	614.1
1:2:4.9	588.9
1:2:[illegible]	4,710.7
Total	28,269.1

The quantities of the various materials required per square yard for the construction of the improvement were as follows:

Cement, bbls.	0.2101
Screened gravel, cu. yds.	0.1316
Sand, cu. yds.	0.0622
Water, gals.	7.0
Oil, gals.	0.2562
Screenings, tons	0.0063

The concrete mixing was done by hand in ½-cu. yd. batches. A crew of 15 men was used. During the first two weeks the efficiency of the gang increased from one batch in 4.2 minutes to one batch in 2.9 minutes, an improvement of 31 per cent. However, this increased efficiency was not maintained throughout the work. The best run was not made until Jan. 20 and 21, on each of which days the batches were mixed in 2.2 mins. After the 21st the work slowed up again. The average time per batch for the whole 52 days was 3.1 mins. As the average working day was 7 hrs. and 40 mins., an average of 4.91 cu. yds. per day per man was maintained. On Jan. 20 and 21, each man mixed 6.97 cu. yds. This 2 cu. yds., the difference between what the gang could do at its best and what it actually accomplished, would have made quite a difference in the contractor's profits. Nor would the men have been rushed, for 2.2 mins. to the ½-yd. batch is only fair work.

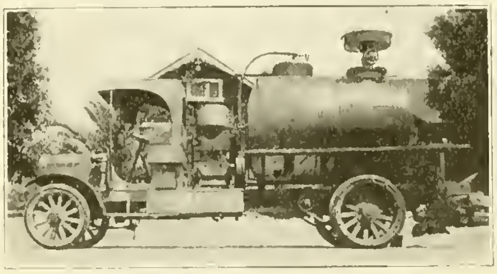


Fig. 2. 1,000-Gallon Tank Mounted on a Motor Truck for Distributing Hot Asphaltic Road Oil.

Samples of the sand and screened gravel were tested daily to determine the proper mixture. Only on one day, Dec. 7, was the amount of sand increased, but the gravel was changed frequently until Dec. 13. After this date the straight 1:2:4 mixture was used until the completion of the contract.

The cost to the contractor of constructing the pavement, not including overhead charges, is as follows:

Item—	Per sq. yd.
Materials:	
Lumber, 26,000 ft. B. M.	$0.0211
Screened gravel, 3,723 cu. yds.	0.0658
Sand, 1,758 cu. yds.	0.0193
Screenings, 180 tons	0.0054
Water, 7 gals. per sq. yd.	0.0005
Total materials	$0.1121
Labor:	
Excavation, 2,997 cu. yds.	$0.0507
Shaping 12,645 lin. ft. roadbed 20 ft. wide	0.0515
Hauling 26,000 ft. B. M. lumber	0.0085
Hauling 3,723 cu. yds. screened gravel 6 miles	0.1126
Hauling 1,758 cu. yds. sand 6 miles	0.0314
Hauling 5,951 bbls. cement 1½ miles	0.0204
Hauling, miscellaneous	0.0029
Mixing and placing concrete	0.0739
Hauling and spreading 172.86 bbls. oil	0.0018
Unloading, hauling and spreading screenings	0.0035
Total labor	$0.3572
Total cost materials and labor	$0.4693

The pavement is 2.39 miles long, 20 ft. wide, and has an area of 28,280 sq. yds. The concrete is 5 ins. thick.

Table I gives the cost to the county of the pavement and of the entire improvement. The figures indicate that little money has been wasted if the wearing coat does not give satisfaction, while a great saving has been effected if it wears well. The total cost of surfacing is very small—0.84 cts. for oil, 0.18 cts. for hauling and spreading, and 0.89 cts. for the screenings in place complete, a total of 1.91 cts. per square yard. The item of heating does not appear in the tabulation, but this is probably included in the cost of spreading. If not, the total cost would not exceed 3 cts.

The cost of the finished road is much less than that usually paid for a first class hard surface pavement, but it is doubtful if the figures could be duplicated on another contract. The cement which was furnished by the county was secured at a very low price. The work was located near the cement mills, and as the manufacturers were conducting a campaign for concrete roads, they made a special price for this contract. The haul was short and over a good road, making the total cost per barrel $1.647 delivered on the work.

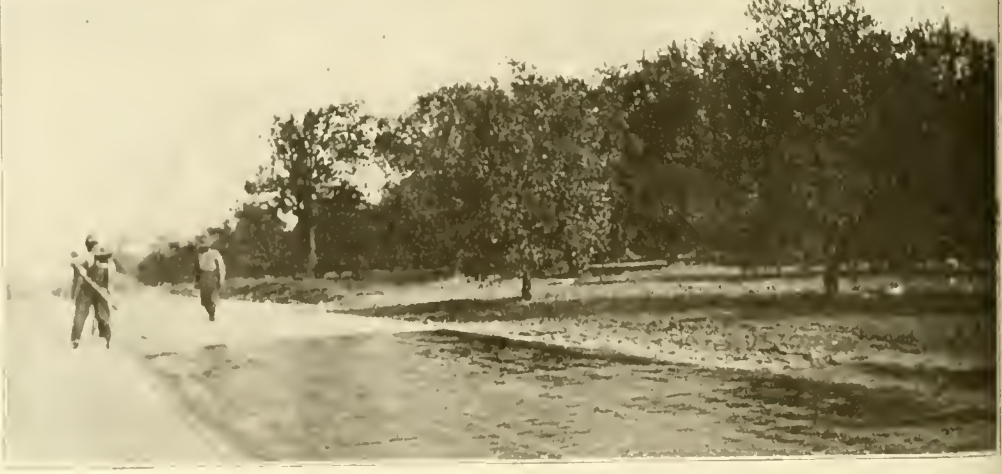


Fig. 3. Method of Spreading Stone Screenings on Hot Asphaltic Oil, Huntington Drive, Los Angeles County, Calif.

The contractor was fortunate if he came out even. The figures show a profit of $3,289.76 on the pavement proper, but the tabulation fails to include his overhead charges, depreciation of plaint, interest on investment and incidentals. As the overhead charges of the county amounted to $1,300, the contractor, whose charges must have been much greater, probably lost money.

For excavation, overhaul and shaping he received $2,732.01 and it cost him $2,891.01, a loss of $159.00. His hauling cost was high, amounting to 37.5 per cent of the total. There is a big difference shown in the cost of hauling sand and screened gravel, the former costing 50.5 cts. per cubic yard, and the latter 85.5 cts. Some difference would be expected, but an increase of 70 per cent is unusual.

If this road wears well, and if the surfacing sticks, many more miles of this type of pavement will be constructed, not only in Los Angeles County, but throughout the state. One thing is certain, however, the contractors will not again bid so low.

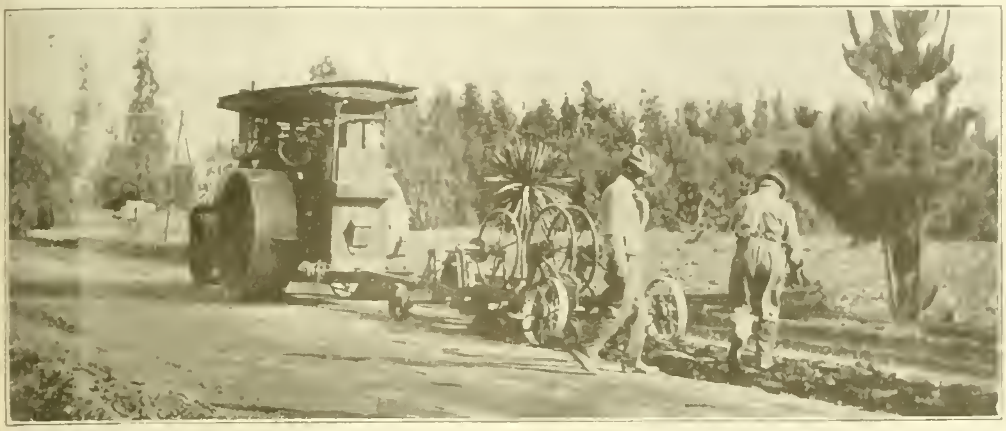


Fig. 4. Plowing Earth Base with a Gang Rooter, Preparatory to Tamping, Huntington Drive, Near Alhambra, Calif.

TABLE I. COST TO COUNTY FOR CONSTRUCTION OF CONCRETE PAVEMENT WITH BITUMINOUS WEARING SURFACE, HUNTINGTON DRIVE, DIVISION 37, SEC. 1.

Item	Unit price.	Amount.	Per sq. yd.
Paid to Contractor.			
Excavat'n, 2,997 cu. yds.	$0.50	$ 1,498.50	$0.0529
Overhaul, 9,546 sta. yds.	0.01	95.46	0.0034
Shaping roadbed, 12,645 lin. ft.	0.09	1,138.05	0.0402
Concrete pavement, 28,280 sq. yds.	0.489	13,828.92	0.4890
Total paid to contractor for pavement proper		$16,560.93	$0.585
Materials Furnished by County.			
Cement, 5,271 bbls.	$1.75	$ 9,224.05	$0.3262
Oil, 172.86 bbls.	1.382	238.98	0.0084
Cost of materials furnished by county		$ 9,463.03	$0.3346
Other County Expenses.			
Inspection		$ 1,086.73	$0.0384
Engineering		263.53	0.0093
County's overhead ch'ges (approx.)		1,300.00	0.0460
		$ 2,650.26	$0.0937
Grates for catchbasins, 6	$6.00	36.00	
Paid to Contractor for Culverts.			
Concrete, class A, 32.88 cu. yds.	$9.00	$ 29[illegible]	
Steel, 1,310 lbs.	[illegible]	[illegible]	
12-in. reinf. conc. pipe, 21 lin. ft.	0.75	16[illegible]	
Total paid to contractor for culverts		$ [illegible]	
Cost to County for Pavement Proper.			
Paid to contractor for pavement proper		$16,560.93	$0.585[illegible]
Cost of materials furnished by county		[illegible]	[illegible]
Other county expenses, except catchbasin grates		[illegible]	[illegible]
Total cost to county for pavement proper		$28,674[illegible]	[illegible]
Cost to County for Entire Improvement.			
Pavement		$28,674[illegible]	[illegible]
Total paid to contractor for culverts		[illegible]	0.018
Grates for catchbasins		36.00	0.0012
Grand total		[illegible]	$1.0[illegible]

Some Service Records of Bituminous Wearing Surfaces on Concrete Pavement.

Service records of bituminous wearing surfaces on concrete pavement are scarce and engineers will read with interest the following notes of conditions found by an examination of the bituminous surfaced concrete pavements of Grand Haven, Mich. These notes are extracted from a report by the W. J. Sherman Co., consulting engineers of Toledo, O.

Sheldon Road.—This is a two-course concrete pavement laid in 1911, with a 1:2 mix for 1½ ins. on top, and 1:2:5 mix for the rest, 6½ ins. total thickness. The top coating was applied in June, 1912. On the last block south from Grant St. there was first applied a coating of ordinary tar with a sand top dressing. This was found not to dry, and was so soft and sticky that it tracked all over. It was removed and a coating of "Pioneer" asphalt was applied to this block, with a sand top dressing. This treatment was also applied to the rest of the street. The block where the tar was first applied is in much the best condition, and there are only a few spots where the top dressing has entirely disappeared, while on the other blocks from Howard St. to Grant St. the coating has now largely gone from the central part of the pavement where the most of the traffic is, while on the sides the coating is in pretty good condition. When there is a hole through to the concrete the coating becomes in some places loosened and may be kicked loose or peeled up in small sections, but it is probable that a large part of the loss of the coating was due to the wear of traffic. The pieces which had been removed were, however, brittle.

[illegible] Street.—This pavement is the same as Sheldon Road, and was laid at about the same time. The top coating is "Pioneer," the same as Sheldon Road. There is very little traffic on this street, and the coating is intact for the most part, there being some places in the center where it has disappeared with one quite large place near Sheldon Road. The difference between the conditions of this street and Sheldon Road is largely due to the difference in traffic.

After these two streets had been used for traffic they were cleaned to receive the top coating simply by sweeping, and it is probable better results would have been obtained had they been washed with a fire hose and allowed to dry, as it is impossible to remove a certain amount of fine dust by sweeping alone.

Lafayette Street, Fourth Street to Seventh Street.—This is a one-course concrete pavement, constructed in the late summer and early fall of 1912. Gravel concrete 1 to 5 mix was laid, making a pavement 6 ins. thick. The top coating was applied in August, 1913, when weather was fair and warm. On the block from Fourth St. to Fifth St. there was first applied a thin coat of common tar, which was put on very hot, and made as thin as possible with a squeegee. The pavement had been washed clean with a fire hose and allowed to dry so that there was no sign of moisture on the surface. The tar coating was applied one day and the top coating of "Pioneer" asphalt was applied the next day, the sand coating of about 1 yd. to 150 square yards of pavement being applied as soon as possible after the asphalt; after this was applied there was placed about ¼ in. to ½ in. of fine sand on top. This was put on to prevent the racking of the asphalt by its sticking to people's shoes while soft. The coating from Fifth St. to Seventh St. was applied in the same way except that tar was put on about 100 lin. ft. and then the asphalt was put on as soon as possible, then two coatings of sand, as in the other block. There was between ¼ and ⅓ gal. of "Pioneer" applied to the square yard of pavement, and this is about all that can be applied, and not have the material run.

The block between Fourth St. and Fifth St. has had some quite heavy traffic, as gravel and dirt were hauled over it for the paving on Fourth St. It is in very good condition, and there seems to be no peeling, and the coating has considerable elasticity. There was no indication of the coating working up through the top coating of fine sand on this block.

That part of the street between Fifth St. and Seventh St. is in poor condition. Almost at once the top coating of fine white sand began to become blackened by the coating coming up through it, and it was soon very black. It would seem as though the tar in this part of the street had either worked up through the asphalt or mixed with it so as to make

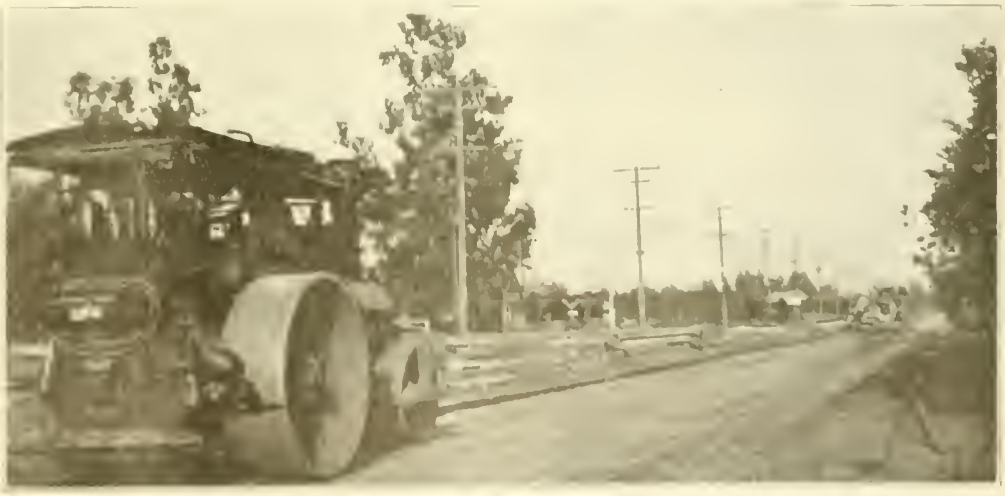


Fig. 5. Rolling Earth Base Preparatory to Laying Concrete, Huntington Drive, Alhambra, Calif.

it so it. The coating on this block seems to have almost no elasticity, and is crumbly. It can be easily loosened from the concrete. The condition of this part of the street has somewhat improved, however, with age, and under the traffic which has not been very heavy. The block from Fourth St. to Fifth St. was the only one which had the heavy traffic referred to above. Surplus sand was removed from both blocks the first week in September.

Second Street.—Six inches 1 to 5 gravel concrete was laid in the fall of 1912 for this pavement. The top dressing of asphalt was applied during the first part of August, 1913. This asphalt was a special preparation made by the Barber Asphalt Paving Co., of Chicago, to comply with the specifications, and on the advice of the Barber Co., no tar coating was first applied. There was about ¼ gal. applied to the square yard of pavement. The pavement was washed with a fire hose and allowed to dry. Coating was applied with a squegee and gone over twice, as only one coating was too thin. Only one top dressing of sand (about 1 yd. to 150 square yards of pavement) was put on. The weather was hot and clear. While all of the coating was applied under the same conditions of temperature of coating, etc., there seems to be great differences of conditions of the coating, which differences cannot be accounted for, as there did not seem to be any difference in the material or method of application. The intersection of Franklin St. and Second St. is in bad shape, there being a great many places where the coating is loose and crumbly without much of any elasticity. About three-quarters of the block from Franklin St. to Clinton St. is in good shape, but there are many places where the coating is loose and crumbly. From Clinton St. to the top of the hill where the grade is steep the coating is in bad shape. The calks on the horses' shoes seem to cut through, and tear the coating loose. On the last block the coating is in good condition and shows practically no signs of soft spots except a few small places which are mostly on the last intersection. This last block is almost on the level.

Wasington Street, Fifth Street to Eighth Street.—Seven inches of 1 to 5 concrete were laid in May and June, 1913. The coating was the same as on Second St. It was applied in the early part of August, in the same manner as on Second St. This is in pretty good condition, though there are some soft and crumbly places. The block from Seventh St. to Eighth St. has a slight grade. It does not show any of these defects, but the intersection of Fifth St. and some parts of the hill between Sixth St. and Seventh St., have some rather bad places.

Seventh Street, Fulton to Washington Street.—Seven inches of 1 to 5 concrete were laid in May, 1913. The top coating was applied in the same manner and of the same material as Washington St. and Second St. This is mostly in good condition, though the street is laid with very flat grades. At the corner of Columbus St. the kettle boiled over and quite a good deal of extra coating had to be removed with shovels as soon as it became a little hardened (in about 2 days). This is the poorest place on the street.

Grant Street, Sheldon Road to Center Street.—Six inches of 1 to 5 concrete were laid in the last of May and early June, 1913. The top coating is the same as Second St. and applied in the same manner. This street has almost no traffic and quite a steep grade. Some rains have washed off quite a good deal of the sand coating, as there had not been enough traffic to drive the sand well into the asphalt. The asphalt here does not appear to have much elasticity and is being cut up somewhat by the horses' calks.

Fifth Street, Washington Street to Howard Street.—Six inches of 1 to 5 concrete were laid in June and the first week of July, 1913. The top coating of "Tarvia A" made by the Barrett Mfg. Co. was applied the last of August and the 1st of September. The street was washed with a fire hose. The coating was applied with a squegee. The first application was thin and the second was made thicker by dragging a squegee back and forth over it. The temperature of the Tarvia was as nearly as possible about 300° F. About ⅓ gal. per square yard was applied and it was found that where applied a little extra thick the material ran a little shown by cracks in the sand top dressing. Sand was spread at about one yard to 150 square yards of pavement. When examined Sept. 13th this dressing was in fine condition, very tough and elastic with no soft or crumbly places.

Howard Street and Fourth Street.—Six inches of concrete 1 to 5 mix were laid August, 1913. "Tarvia A," about ¼ gal. to the square yard was applied in the same manner as on Fifth St. The concrete on the block from Washington St. to just south of Franklin St. had only been laid for a week to two weeks before the top coating was applied, the intersection of Franklin St. being just one week old. This last block and a little over was never thrown open to traffic before the coating was applied, and was consequently rougher on top. This was not washed with hose, but was swept very clean. The weather had been hot, and the top of the concrete seemed quite dry. As the top dressing was applied it seemed to stick to concrete much better than where the surface had been worn smooth by traffic. This street was all in fine condition similar to Fifth St. when examined Sept. 13.

Washington Street, Fourth to Fifth Street. The concrete 1 to 5 mix was laid 7 ins. thick. Part of this street is 52 ft. wide, but most of it has two 18 ft. roadways with parking in the center. The north half of the street was laid in July, the south half was laid in the latter part of August, being from 10 days to 2½ weeks old when the top dressing was applied. The north half of the street had been opened to traffic and was washed by a fire hose. The south half was not opened to traffic and was rough like part of Fourth St. It was, however, washed with a fire hose. "Tarvia A" was applied the same as on Fourth St. and Howard St. This was all in fine condition when examined in September.

Concrete Road Construction—A Symposium from Committee Reports to the National Conference on Concrete Road Building.

Fairly representative engineering opinion on preferable methods and practice in the design and construction of concrete roads is given by the reports of committees to the National Conference on Concrete Road Building, which was held in Chicago on Feb. 12, 13 and 14. There were 15 committees appointed to report and of these 11 presented reports. Such of these reports as are available and of most general interest are summarized as follows:

METHODS AND COSTS OF REPAIRING AND MAINTAINING CONCRETE ROADS.

Two kinds of repair work are performed on the concrete roads of Wayne County, Michigan; one is repairing cracks and pits and one is resurfacing the whole pavement top. For repairing of the first kind a crew consisting of seven men and a team, provided with a tar kettle, is utilized. The foreman is paid $5 a day, the team and driver $5 a day, the "tar man" $3 a day, two laborers at $2.50 each and two laborers at $2.25 each. The tools used consist of two wire bristle brooms, a wheelbarrow, a couple of shovels and a tar bucket with a round spout. Tarvia X is now used exclusively. A lighter grade was first tried but did not give such permanent results as the heavier grade. Two men are utilized to sweep all cracks clean with the wire brooms, after which the man with the tar kettle fills the cracks with the tar which is heated to about 225° F. An excess of tar is poured in so that it extends an inch or so beyond the edge of the crack. It is then allowed to stand in the crack for a few minutes to prevent it from "bubbling" out in case the sand is wet. The sand, which should be dry and coarse, is spread with a shovel over the crack and into the tar, and the whole is left for traffic to iron out. The excess tar and sand are worn away rapidly, leaving a smooth, even surface, over which no jolt is apparent in passing either with a horse drawn or motor driven vehicle. This method of repair prevents the edges of the concrete from spalling and chipping, and no water can get through to the subgrade to freeze and heave in cold weather. The work is preferably done on hot, dry days. It has been suggested that the better time to handle this work would be in the late fall when the cracks would be open the widest due to contraction, but the results we have secured in the summer months have been so satisfactory that we have not tried out the latter plan. The small pit holes which are due simply to some foreign substance getting into the concrete like clay, wood or some fragment of inferior rock which might chance to be a part of the aggregate, are treated in the same manner. The cost of this kind of repair work for the year ending Sept. 30, 1913, was as follows:

Item.	Cost.
71 bbls. tar at $6	$ 426.00
32½ days labor (gang of seven) at $22.50	731.25
66 cu. yds. sand at $2	132.00
Total	$1,289.25

To this total there was added to cover engineering, inspection, depreciation of plant, overhead charges, etc., a sum of approximately $1,450. These costs cover the repairs to 51 miles of road.

The second kind of repair work, that is treating the entire surface of the road where the concrete for any cause has not withstood wear has not been so successful as the first. One-half mile of concrete on Fort Road, about 800 ft. on Grand River Road, and 1¾ mile on Gratiot Road, out of the 80 odd miles of concrete road in Wayne County, are not up to the standard of the balance of the construction, and they have been surfaced with Dolarway, a special grade of Tarvia and Tarvia X. The surfaces of Fort and Gratiot Roads were both rough, having been built in cold weather, and opened for traffic before the concrete had thoroughly hardened. Fort Road was surfaced by the contractor before the Board would accept the road and make the final payment. It carries a fairly heavy mixed traffic, not as great, however, as either Grand River or Gratiot Roads. A special grade of Tarvia was used. The road was first swept by a street sweeper which was followed by men with wire brooms. About ½-gal. of bitumen to the square yard was used, and the whole covered with coarse sand and rolled with a 10-ton roller. This repair was made early in the summer and was not touched in any manner for a period of two years. By this time it had scaled off in spots quite badly, and was gone over again in the summer of 1913, using Tarvia X and a washed roofing gravel about ¼ in. in size. We have no record of what the first cost was to the contractor, as we paid nothing for it.

Gratiot Road was surfaced at cost to us by the Dolarway people in the spring of 1912. By the summer of 1913 about half of the original coating had scaled off, at which time we patched it up, using a special grade of Tarvia and Tarvia X. The Tarvia X has given the better result to date. Washed roofing gravel was used with the bitumen. This work was also rolled with a 10-ton roller. Gratiot Road carries an average traffic of over 1,000 vehicles a day. (On the last count made one week in August and one week in September the average traffic per day was 1,210, of which 36 per cent was motor traffic and 64 per cent horse-drawn vehicles.) The surface of the road is wavy, and we have found it necessary to go over the work two or three times during the summer, touching up small spots here and there where it had scaled off. The cost of the work done by our Board is in the neighborhood of 9 cts. a square yard.

The committee making the report from which the above extract is taken consisted of E. N. Hines, chairman, Board of County Road Commissioners, Wayne County, Michigan; J. S. McCullough, City Engineer, Fond du Lac, Wis., and F. P. Wilson, City Engineer, Mason City, Ia.

PREPARATION OF SUBGRADE FOR CONCRETE ROADS.

The fundamental requirement of the subgrade is that it shall at all times be of uniform density, so that it will not settle unevenly and cause cracks in the concrete surface. Some engineers apparently believe that it is not necessary to take as much care in preparing the subgrade or foundation of a concrete pavement as of other forms of roads having an artificial surface, because the concrete slab will act as a bridge over any soft streak or low spot in the foundation; but the committee is of the opinion that this is a mistake. The strength of a plain concrete slab in acting as a beam to carry the load over a low spot or soft place in the foundation is very slight; and it is so easy to remove the low place or soft spot as not to justify the dependence upon the beam action of the concrete. Any uneven settlement of the foundation of a concrete pavement is nearly certain to cause a crack. With most forms of pavements a crack in the surface will heal under traffic; but a crack in a concrete pavement not only can not heal under traffic, but will continually enlarge. There is no part of the work of the construction of a concrete pavement that is more worthy of intelligent care and painstaking labor than the preparation of the subgrade; and the slight additional cost necessary to insure good results is abundantly justifiable.

If the concrete pavement is to be constructed upon an old roadbed of any kind, either an earth or a broken stone or gravel road, great care must be taken in preparing the subgrade. The old roadbed is likely to be more compact in the center than at the sides; and consequently there is danger that the concrete pavement will settle more at the sides than at the center, and therefore will crack longitudinally. Further, it is likely that the traveled way of the old road will not at all places be central under the new concrete pavement, and consequently the latter will settle unevenly and crack.

When the subgrade is an old roadway, it is not sufficient to roll the subgrade longitudinally, since the roller is likely to balance upon the more compact central core, and therefore not consolidate the soil at the side of the old roadway. It is not necessary to attempt a detailed description of the method of overcoming this difficulty; but the engineer should be alert to determine whether this condition obtains, and when it does occur he should take the necessary precautions to secure a thorough consolidation of all parts of the new roadbed. It may be necessary to add material at the side of the more compact central core of the old roadway, or it may be wise to roll the roadway diagonally or perhaps transversely. In extreme cases it may be necessary to loosen the old roadbed by spiking or scarifying, and then harrow it, and finally consolidate the entire new roadbed with the roller. The Committee desires to emphatically assert that the need of care in this matter is not imaginary, and that such conditions do really occur in actual practice.

The drainage of the roadbed of a concrete pavement is of vital importance. If the subgrade is not well drained, there is danger that after the concrete is laid, the drying of the soil under the edges of the concrete may permit the pavement to settle and thus cause longitudinal cracks on the surface. Further, if the subgrade is not well drained, there is a possibility that the frost may lift the edges of the concrete roadway and cause a longitudinal crack, at least on the lower side and possibly also upon the upper surface of the concrete.

If the soil is sandy, there is a possibility that the natural under-drainage is sufficient for the purpose.

If the soil is only moderately retentive, it will probably be sufficient to lay a line of ordinary farm tile along each side of the concrete pavement. The tile should be at least 4 or 5 ins. in diameter, should be laid $2\frac{1}{2}$ to 3 ft. below the surface, and should have sufficient fall to free itself promptly and fully. It is better to lay the tile outside of the edge of the concrete slab than under it. If the soil is a little retentive, some engineers put a layer of coarse gravel or broken stone immediately above the tile to facilitate the entrance of the water into the tile.

If the soil is quite retentive, it is wise to make more elaborate provisions than above for the drainage of the subgrade. For this case, it is recommended that a shallow longitudinal ditch be constructed just outside of the edge of the concrete slab. This ditch should extend about 8 or 10 ins. below the surface of the roadbed; that is, below the bottom of the concrete slab; and should be filled with coarse gravel or broken stone. From this longitudinal ditch short transverse ditches should be dug across the shoulder to the ditch at the side of the roadway. These transverse trenches should have a grade sufficient to permit them to carry the water promptly and fully to the side ditch. In particularly retentive soil, these transverse trenches should not be placed more than 50 ft. apart. On level stretches, these transverse ditches should be practically at right angles to the direction of the road; but if the road is on a grade, these trenches should make an acute angle with the roadway, the amount of this angle depending upon the grade of the road. The sloping of these lateral trenches down hill makes it unnecessary to have the ditches at the sides of the roadway so deep as would otherwise be required. These lateral ditches should be filled level full with broken stone or coarse gravel to a point at least a little beyond the outer edges of the shoulders and preferably nearly to the bank of the ditch at the side of the roadway.

The committee from whose report the above extract is taken consisted of Ira O. Baker, Professor, Civil Engineering, University of Illinois; A. R. Hirst, State Highway Engineer, Wisconsin, and A. N. Johnson, State Highway Engineer, Illinois.

CONCRETE AGGREGATES FOR CONCRETE ROADS.

The essentials of aggregates for concrete for roads are:

(1) For fine aggregate, use only sand or other fine aggregate that has been actually tested for mechanical analysis and tensile strength of mortar, and is free from fine particles.

(2) Use coarse-grained sands or hard stone screenings with dust removed.

(3) Use sand or other fine aggregate that is absolutely clean.

(4) For coarse aggregate, use hard stone, such as granite, trap, gravel, or hard limestone.

(5) If bank gravel or crushed stone is used, always separate the sand or screenings and re-mix in the proper proportions.

If local conditions prevent following any one of these rules, adopt some other material than concrete for your pavement.

(1) Actual laboratory tests are necessary for fine aggregates, because it is impossible for the most expert builder always to distinguish by appearance between good and poor sands. Sand may be coarse, of good color, well graded, and apparently perfectly clean, and yet because of a minute quantity of vegetable matter may show practically no strength when made into mortar or concrete. Case after case has been found where good looking sand had to be rejected on laboratory test or, if used, produced defective concrete.

(2) Coarse sand is necessary not only for strength and density, but to prevent the formation, on or near the surface, of a layer of fine material, consisting of a mixture of dust and cement which has no durability. Mortar made with fine sand or sand having a large proportion of fine grains of silt, hardens slowly and is especially objectionable in cold weather. This prevents it attaining proper strength before the road is thrown open to traffic. A sand having a considerable proportion of fine particles may possibly show high briquet tests, and yet the mortar not have good resistance to attrition or wear.

(3) Sand must be absolutely free from vegetable or organic matter, or it is liable to harden not at all or too slowly to be serviceable. Frequently, sand may be entirely satisfactory in appearance, and yet be worthless for concrete. Defective sand of this type is apt to be taken from too near the surface of the ground, so that it contains a very small percentage of vegetable loam. At least 2 ft. of top soil and loam should be removed before using the sand, and in many cases it is necessary to take off as much as 4 or 5 ft. while occasionally no acceptable sand can be found in the entire bank because of the penetration to a great depth of the deleterious vegetable material.

(4) A coarse aggregate of hard quality is necessary to resist the wear and abrasion of hoofs and wheels. Failures of concrete roads have been caused simply by the softness of the coarse aggregate. In one instance, for example, shells were used for the aggregate, and the road went to pieces as soon as it was subjected to wear. All stone, like shale, slate, shells, and soft limestone, must be rejected; while trap, granite, and conglomerate, are especially suitable materials. A hard limestone, such as that occurring in certain localities along the Hudson River, which is sold in New York as trap rock, is satisfactory for concrete roads. A hard limestone cannot be cut with a knife and the specific gravity is high, say, over 2.70.

Gravel does not bond quite so strongly with cement as does broken stone. When properly screened and free from dirt, however, and remixed with sand in the proper proportions, a good surface can be made for a one-course pavement.

(5) Many roads that are now being built will prove worthless because of the use of sand taken directly from the bank without screening. If the gravel contains as much as 40 per cent of stones and very rich proportions are used, say one part cement to $3\frac{1}{2}$ parts bank gravel, a fair concrete can sometimes be produced, but it is always cheaper in such cases to screen the gravel and remix the sand and stone in proper proportions. There will be, for example, a saving of $\frac{1}{4}$ barrel, or 1 bag, of cement per cubic yard of concrete by using proportions 1 part cement to 2 parts sand to 3 parts screened gravel, instead of using the unscreened bank gravel in proportions $1:3\frac{1}{2}$. This difference will more than pay for the additional cost of screening the sand and rejecting part of it. At the same time, the result will be more uniform and the surface more durable because of the stones which take the wear. When an excess of sand is used in the mixture, as is the case with run-of-the-bank gravel, the mortar rises to the top when the concrete is placed and the wearing surface is less resistant than a mix that is uniform throughout.

The committee from whose report the above extracts are made consisted of Sanford E. Thompson, Consulting Engineer, Newton Highlands, Mass., A. N. Talbot, President American Society for Testing Materials, and W. M. Kinney, Assistant Engineer, Universal Portland Cement Co.

ECONOMIC METHODS OF HANDLING AND HAULING MATERIALS FOR CONCRETE ROADS.

The materials to be handled in the construction of a concrete road are: (1) Water, (2) cement, (3) sandstone, and (4) gravel or stone.

Water. The first mentioned material, water, is quite apt to be given scant consideration by the road builder whose experience has been limited to work within cities or villages. Likewise the engineer or contractor who has never before attempted road construction is prone to underestimate the cost of securing and delivering water, whether for concrete or macadam roads in country districts. In arid regions a mistake of this sort may readily result in serious financial loss.

The chairman of your committee has been personally interested in the building of concrete roads under conditions that have served to emphasize the importance of considering the cost of water and of ways and means to reduce that cost.

Water, it should be remembered, is used for at least three purposes in building concrete roads: (1) To wet the sub-grade; (2) to mix the concrete; and (3) to keep the concrete moist for several days after it is laid. On a sandy sub-grade that must be wet and rolled before laying the concrete, as much as 100 gals. of water per cubic yard of concrete may be used merely to wet the sub-grade. Then an additional 50 gals. or more per cubic yard will be required to mix the concrete wet enough to flow readily. Finally, as much as 80 gals. per cubic yard of concrete will be needed to keep the concrete wet for a week after it is laid if the air is very dry and if there is much wind. A total of 230 gals. of water, weighing 1,900 lbs., has been required for each cubic yard of concrete, in one case known to your chairman. More than that would have been used had the concrete not been kept covered with a layer of earth an inch or more thick. The earth, of course, retarded evaporation of the water. The water in this case was hauled in tank wagons, so there was no guessing as to the total amount used, although its distribution among its three different uses was not accurately determined.

A ton of water per cubic yard of concrete may probably be regarded as the maximum that need ever be required. A quarter of a ton of water per cubic yard is probably the minimum, if there is to be any sprinkling of the sub-grade and of the concrete. A two-mile haul at 25 cts. per ton-mile results in a cost of 50 cts. per cubic yard of concrete for hauling the water where the maximum amount is needed. This is astonishing indeed to the contractor who has been accustomed to mix and place concrete for 50 cts. a cubic yard under certain conditions.

When large quantities of water are to be moved considerable distances pipe lines should usually be laid along the road and the water should be pumped. Quite a common mistake, when this is done, is to use pipe of such small diameter that the friction head makes it impossible to deliver the quantity of water needed. Thus a 1-in. pipe is often laid where a 2-in. pipe is needed. Any engineer, however, is competent to estimate the size of pipe required under given conditions. It should be remembered that the head against which the pump is acting is the sum of the static head and the friction head, and that the cylinder of a pump may be cracked under an excessive pressure thus developed. Within the confines of a report of this character it is impracticable to discuss at further length the problem of water supply for concrete road work.

Cement.—In the handling of cement there are two very common economic errors in evidence: (1) Failure to provide a large stock of cement, and (2) an attempt to locate the cement nearer the concrete mixer than is necessary. The concrete road builder is so often forced to wait for the delivery of cement by rail that he eventually learns that it pays to tie up more capital in a stock of cement than to foot the bills incident to delays in securing it by rail. Although cement dealers are usually prompt in making shipments, the railways are not always to be relied upon. For this reason, cement should be accumulated in stock-houses or stock-tents Tents, with wooden floors made in sections, can be placed at short intervals along the road. From them the cement can be delivered in small gasoline motor cars of the kind that cost about $500 each. Such a car will carry 10 or 12 bags of cement, and can be run into the tent to be loaded by the driver of the car. The car will travel 15 to 20 miles an hour loaded and 18 to 25 miles an hour empty, at a total cost of about 4 cts. per car-mile one way or 8 cts. per car-mile of effective hauling work. The resulting cost of hauling cement short distances can not be equaled in any other way known to your committee.

Sand. After the sub-grade has been sprinkled and compacted by rolling, it is customary to dump sand and gravel on it in piles. Then these materials are loaded into barrows and wheeled to the concrete mixer which travels along the sub-grade, leaving the layer of "green" concrete behind. This, we think, is at present the most common way of placing concrete. But it is not the only way, and there are some contractors who maintain that it is not the cheapest way. Another method is to place both the mixer and the stock piles on one side of the road. The materials are then usually shoveled directly into the mixer, or into a bucket or skip that delivers into the mixer. In the West it is not unusual to see Fresno scrapers used to drag the sand and gravel from the stock piles to the mixer. In some cases the scrapers deliver the materials upon a platform whence they are fed by gravity into a skip that delivers into the mixer. In these cases the mixer is not moved until about 600 to 1,000 ft. of road have been built in both directions from the mixer, the concrete being hauled in one-horse concrete carts or in one-man concrete buggies.

The differences in cost between these two methods seem ordinarily not to be very great; but there may be conditions that justify a choice of one method in one place and of another method in another place. Thus, if laborers are scarce and mules or horses plenty, the latter method may be preferable.

As above stated, a small $500 motor car can be economically used to haul cement short distances. To do this the body of a pleasure car is removed and a platform placed on the rear part of the chassis. This method of hauling cement was first devised by one of your committee, it is believed; and he now suggests that the same method might be advantageously applied in hauling concrete from a mixer to the road. To do this a concrete bucket would be mounted on the chassis, and the concrete would be discharged from the bucket through a trough. It might be desirable, in using such a method, to provide a pair of trussed planks that would span the road, and be used as a runaway for the automobile.

So much has been written about the unloading of cars of sand, etc., with drag scrapers, clam-shell buckets, and the like, that we shall not here go into these labor-saving methods. An ingenious modification of old methods was recently described. It consists in dumping sand or gravel from a hopper-bottom car into a pit dug beneath the track. The materials are thence dragged with a power-scraper up an incline and dumped into a bin. This method has much to commend it where hopper-bottom cars are available.

Gravel and Stone.—Although cement and even sand may be rehandled without adding greatly to the cost of a concrete road, an attempt should always be made to avoid rehandling the gravel or broken stone. Broken stone is particularly hard to shovel up when dumped on the ground; and, besides, it constitutes such a very large part of all the materials to be handled. Stock piles of sand may often be needed to avoid delays in freight shipments; and, in such cases, they may be placed at relatively short intervals along the side of the road. If broken stone or gravel is also received by rail, it is generally wise to build the stock pile at the place where the cars are unloaded. Then a clam-shell bucket can be used both to unload the cars into the pile and to load the wagons from the pile.

If large stock piles of stone or sand are to be made along the road, care should be taken to scrape a smooth place upon which to build the pile. With a road machine and a roller a place can be so graded and compacted that very little of the material in the stockpile will be lost. Care should be taken to locate such stockpiles where heavy rains will not wash the materials away.

In the handling of broken stone from portable rock crushers, two economic errors are common: (1) Bins are too small, and (2) their bottoms are not steep enough. Small bins make it necessary to stop the crusher if there is not perfect co-ordination in hauling and using the materials. Bottoms that are flatter than 1 to 1 prevent the stone from running rapidly, and thus delay the loading This last is important enough with team-hauling, but it is much more important where motor trucks or traction engines are used.

Hauling by Power.—Although the horse and the mule will long continue to be used on road work, their "day" for hauling road materials is fast passing. We have already given some details relative to the use of small motor cars for hauling cement short distances. Motor trucks of 3-ton to 5-ton capacity are becoming popular for hauling wherever the roads are sufficiently good to admit their use. Recently the use of "trailers" has added much to the popularity of motor trucks. While a "trailer" decreases the speed of transportation it greatly increases the tonnage hauled each trip.

Traction engines hauling trains of wagons have been in successful use for so many years that we need say little about them.

As to the relative economic merits of motor trucks and traction engines, no data yet available point either to the one or the other as being clearly preferable. In making comparisons, several things other than the ton-mile costs while hauling during the road building season should be kept in mind: (1) Whether the hauling plant can be used for other purposes than road building; (2) whether it can be used in the winter; (3) whether it destroys existing roads and sub-grades sufficiently to be objectionable; (4) whether there is room to maneuvre—turn around, etc.

Not a few men are making it their sole business to contract for hauling. Such men often work their hauling plant two shifts daily, running about 18 hours out of the 24. This greatly reduces the interest cost per ton-mile. A not uncommon price is 15 cts. per ton-mile over rough roads and under severe conditions.

Conclusion.—Your committee has made no attempt to discuss concrete mixing and placing, for there are fully a dozen books in which this matter is given much space. Nor has it seemed wise to repeat much that also exists in book form on the general subject of shoveling and hauling. A few suggestions as to recent developments in concrete road work have been indicated in the hope that other road builders will add their quota.

The committee from whose report the above extracts are made consisted of Halbert P. Gillette, Editor-in-Chief, ENGINEERING AND CONTRACTING; Donald D. Price, State Engineer, Nebraska, and Percy H. Wilson, Secretary, Association of American Portland Cement Manufacturers.

MIXING AND PLACING CONCRETE FOR CONCRETE ROADS.

Mixing Concrete.—The concrete mixer should be of the batch type, provided with an automatic water tank, traction drive and power loader. It is recommended that all specifications contain a clause to the effect that all the materials in any one batch, including the water, should remain in the drum of the mixer at least 45 seconds before any of the concrete is discharged.

Placing Concrete. Just before placing concrete, the sub-grade should be well sprinkled so that it will not absorb moisture from the concrete. Probably the most satisfactory method of depositing the mixed concrete in position on the sub-grade is by means of a bottom dump bucket running on a swinging boom free from the mixer. The boom can be swung over any position on the sub-grade, and the bucket can be run out and dumped at any point along the boom. Such equipment does not depend for economic operation upon consistency and the concrete can be easily handled and deposited at any place on the sub-grade, necessitating but little handling with shovels. Between each change in position of the mixer a section the full width of the pavement (up to 1½ times the length of the boom, and as long as the boom), may be concreted.

When concrete is placed by means of the open trough care must be exercised to see that it is not mixed too wet, as this method of distribution depends for economic operation very much upon the consistency of the concrete. An open trough is necessarily much shorter than a boom and is more limited in action, which requires that the mixer be moved ahead more often, also that more of the concrete after being deposited on the sub-grade be handled with shovels.

When a batch mixer, not having a boom and bucket or open trough, is used, the concrete should be handled in wheelbarrows or hand carts wheeled on suitable runways. Placing concrete by means of horse carts should not be permitted, as they cut up the sub-grade badly and to get the concrete into final position requires a maximum amount of shoveling.

Whatever the method of conveying and placing, the concrete should be deposited upon the sub-grade to the required depth and for the entire width of pavement in as nearly one operation as practical so as to reduce handling to a minimum. If it is necessary to handle a large amount of material with the strike-board it will ride the concrete and thus produce a wavy, uneven surface. Attention to this matter will materially lessen the difficulty and labor in striking off the surface and will result in a very much better finish.

The excess of coarse material that accumulates in front of the strike-board should be uniformly distributed over the surface of the pavement and not left in narrow strips across the section, or placed along transverse expansion joints. An ordinary garden rake will be found useful in distributing the material that accumulates in front of the strike-board. When the strike-board is within 2 ft. of an expansion joint the excess material that has accumulated in front of it should be removed with a shovel and deposited on the sub-grade in the next section. Special care should be taken to place good dense concrete along expansion joints, and all inequalities at the joints, including any small holes left after the removal of the installing device, should be filled with a mortar composed of 1 part cement and not more than 2 parts fine aggregate.

The man handling the strike-board should follow closely behind those placing the concrete, for keeping up with the strike-board will materially assist those placing concrete in depositing the required amount of material.

The placing of concrete should be a continuous operation, and stops should be made only at expansion joints. In case the mixer breaks down concrete should be mixed by hand to complete the section, or an expansion joint should be placed at the point of stopping work. Any concrete in excess of that needed to complete the section, when work is discontinued, should be spread out in a thin layer, not exceeding 2 ins. in thickness, over the sub-grade in the next section and not piled up along the expansion joint.

The strike-board should be cut to conform to the crown of the finished surface of the pavement and should be of sufficient strength and stiffness to show no deflection at the center when supported at the ends, nor a material bowing out of alignment when in use. It should be about 2 ft. longer than the width of the road, protected on the bottom edge with a metal facing and provided at each end with suitable handles.

For roads up to 12 ft. in width two 2x6-in. planks, dressed on one side and both edges, spiked together, make a good strike-board, and for roads 12 ft. to 20 ft. in width two 3x8-in. or a 2x10-in. and a 3x10-in., spiked together, will be found satisfactory. Two planks, well spiked together, make a better strike-board and one less likely to warp out of shape than a single piece. The strike-board should not be so heavy that it cannot be easily handled by two men, and to reduce weight and increase stiffness, it is advisable to work over 20 ft. in width to use a strike-board composed of boards 2 ins. or 3 ins. thick, stiffened by trussing, rather than obtain the required stiffness by the use of heavier timbers

In striking off a pavement 16 ft. or less in width, where the concrete has been properly placed, two men should be able to handle the strike-board, but for wider pavements the services of a third man will be required to assist in pulling the board forward by means of a drag line fastened at its center. The strike-board should always be worked forward about perpendicular to the axis of the roadway and as it is moved ahead should be sawed back and forth across the road. To produce the desired effect will require that the strike-board be passed over the surface of the concrete two or more times.

Though it will be necessary for the workmen to get into the concrete some after it is deposited on the sub-base, it is desirable that all unnecessary walking in and wading through the concrete be avoided, and under no circumstances should any workmen step upon the concrete after it is first struck off.

The committee from whose report the above extracts are taken consist of Paul D. Sargent, Chief Engineer, State Highway Commissioner of Maine; A. H. Blanchard, Professor, Highway Engineering, Columbia University, and E. W. Boynton, Inspecting Engineer, Universal Portland Cement Co.

COST OF COSTRUCTING CONCRETE ROADS.

The average cost reported in several different states, comprising the data to one-course construction, is as follows:

State.	Cost per sq. yd.	State.	Cost per sq. yd.
Arizona	$1.20	Montana	$1.83
Arkansas	.90	Nebraska	1.40
California	1.17	New Jersey	1.12
Colorado	1.25	New York	....
Connecticut	1.32	North Dakota	1.26
Delaware	1.61	Ohio	1.22
Idaho	1.09	Oklahoma	1.05
Illinois	1.01	Oregon	1.39
Indiana	1.23	Pennsylvania	1.16
Iowa	1.11	Tennessee	1.20
Kansas	1.08	Washington	1.31
Maine	1.48	West Virginia	1.32
Maryland	1.21	Wisconsin	1.06
Massachusetts	1.29		
Michigan	1.27	Average total	$1.24
Minnesota	1.05		
Missouri	1.17	Weighted tot. av.	$1.19

The committee from whose report the above data was taken consisted of A. N. Johnson, State Highway Engineer, Illinois: Joseph H. Pratt, State Engineer, North Carolina, and Albert Reithmann, Past President, Western Society of Engineers.

THICKNESS, CROWN AND GRADES FOR CONCRETE ROADS.

Thickness.—The thickness of the concrete pavement is controlled by condition and character of the sub-grade, drainage, traffic, climatic conditions, width of pavement, etc. Three distinct types of cross-sections are in general use:

(A) Uniform thickness of concrete for all widths of roadway, and consequently with the same amount of crown in the foundation as in the surface.

(B) Roadways in which the concrete is thicker at the center than at the edge but in which some crown is given to the foundation.

(C) Roadways in which the concrete is thicker at the middle than at the edge, but which are built upon a flat sub-grade. This type seems to be fast gaining in favor.

(1) Where the width of roadway is not greater than 16 ft., with a porous sub-soil, sub-grade in good condition and well packed and with loads not exceeding six tons, including the vehicle, a thickness of 5 ins. at the edge of the slab and 7 ins. at the center is sufficient without reinforcement, using slabs not greater than 16 ft. square.

(2) On clay, soil and the above general conditions artificial drainage should be provided and reinforcement used (¼ to ½ lb. per square foot), placed 1½ to 2 ins. from top surface; and slabs made not larger than above and not less than 6 ins. thick at edge of slab and 8 ins. at the center.

(3) Where the roadway is in excess 16 ft. with loads of 6 to 10 tons, but with the other general conditions as described in (1), the crown of the pavement should have a thickness of 8 ins. and the edge 6 ins. Under such circumstances reinforcement will usually be considered necessary and economical

Crown.—The small crown that may be properly used on concrete pavements is justly considered an important merit of this type of pavement. Data have been collected which seem to show that in the case of uniformly thick concrete pavements the amount of central longitudinal failures has varied directly with the amount of crown. To a lesser extent this would be expected of the other types thus furnishing an additional reason for the adoption of a minimum crown.

From data of practice in about 30 cities it appears that the minimum crown used on streets is 1/160 of the width of the road and the maximum 1/48. With few exceptions, the crown varies between 1/70 and 1/100 of the width. When the climatic conditions will allow, it is our opinion that a crown of 1/100 of the pavement width represents present best practice. The use of the high crowns has in large part resulted from past experience with other types of pavements where high crowns were really required. We are of the opinion that in future construction, as methods of handling materials are improved and contractors become more experienced, engineers should not hesitate to specify as low a crown as 1/150 part of the width. Indeed, this has already been done to some extent.

The present common practice of specifying an arbitrary crown ratio for streets varying in width by 75 per cent or more, and this, too, quite independent of the street grade, does not appear to be founded on any reasonable basis. Unless local conditions forbid, a smaller crown should be designed for a heavy than for a light grade street. Tables have been published which give the allowable crowns as a function of the grades. This whole matter would seem to deserve much more attention than it commonly receives.

Grades.—From such observations as we have been able to make, we see no reasons why a concrete pavement, properly finished, may not be used on grades as steep as any other hard surfaced pavement, and on steeper grades than either wood or asphalt. On steeper grades than 3 or 4 per cent shallow corrugations in the surface will be found of much assistance—in fact, with grades of 5 per cent and over, a real necessity.

The committee from whose report these abstracts are taken consisted of L. C. Smith, University of Wisconsin; E. R. Whitman, City Engineer, Port Huron, Mich., and T. R. Agg, Iowa State College.

PROPORTIONS AND CONSISTENCY OF CONCRETE FOR CONCRETE ROADS.

Any specifications for proportion and consistency of concrete for roads can only be general and suggestive.

Base.—For sound hard limestone or gravel having 40 per cent voids, screened washed bank sand having 30 per cent voids and Portland cement, passing the standard specifications of the American Society for Testing Materials, the proportions should be 1 sack of cement, 2½ cu. ft. of sand and 5 cu. ft. of crushed stone or gravel, the aggregate having been so proportioned as to eliminate the voids as far as practicable.

The amount of water should be such as to make the concrete plastic and still retain its shape, such as is commonly called a quaking mixture. Methods and distance of transportation before depositing will be factors in determining the amount of water used; concrete deposited from buckets, traveling along a boom, may have more water added than when wheeled for a considerable distance in barrows or carts. In the latter case the unavoidable shaking brings the lighter materials to the surface and causes a segregation of the materials, which results in an improper mixture when deposited.

Wearing Course.—The fine aggregate for wearing course shall be mixed with Portland cement in the proportion of 1 sack of cement to 2 cu. ft. of the graded mineral aggregate. While this furnishes a considerable excess of cement we get greater impermeability.

Sufficient water shall be used with the fine aggregate to form a mortar that will work easily under the template and at the same time retain its shape when deposited.

The committee consisted of C. U. Boley, City Engineer, Sheboygan, Wis.; C. C. Widener, City Engineer, Bozeman, Wis., and Geo. [illegible] Dingman, Wayne County, Mich., Road Engineer.

Power Plants

Construction Features of the Intake Dam of the Tallulah Falls Development, for the Georgia Railway and Power Co. in Georgia.

Contributed by Charles Adsit, Engineer Northern Contracting Co., and Eugene Lauchli, Hydraulic Engineer.

This structure, of gravity section, is one of the three dams forming part of the 100,000-H.P. hydro-electric development now practically completed for the Georgia Railway & Power Co., at Tallulah Falls, Ga., which has been put in service during the past summer months, and, on account of its location and duty is known as intake or diverting dam. The dam serves the double duty of adding 100 ft. to the natural head of 500 ft. on the plant and of raising the water to the 12x14-ft. diverting tunnel which has been driven through the solid rock, 6,665 ft. to a forebay or surge reservoir, at the head of the penstocks. It also forms a storage basin 1½ miles long, the capacity of which is 106,000,000 cu. ft., 60,000,000 cu. ft. being available for the immediate demand of the water wheels.

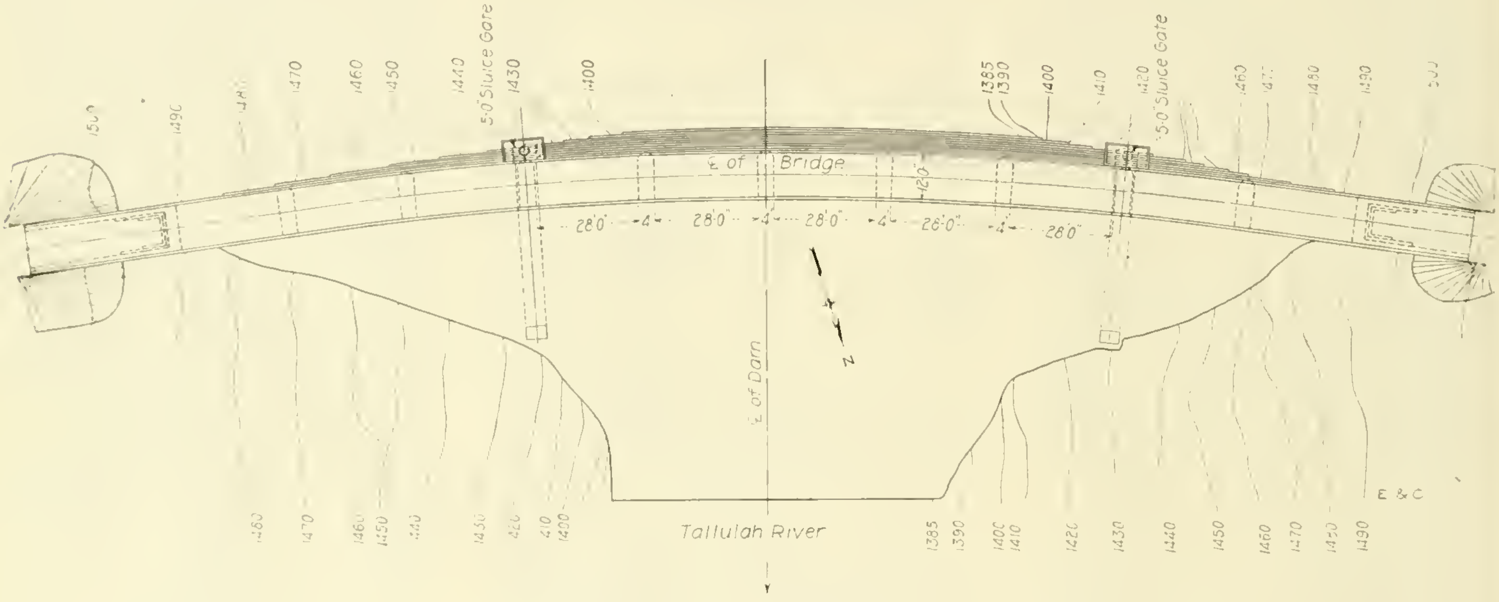


Fig. 1. General Plan of Concrete Intake Dam, Tallulah Falls Hydro-Electric Development.

The drainage area above the dam is 192 square miles, most of which is well timbered, and with steep slopes, and the annual rain fall varies from 70 to 80 ins. During construction, the river flow has varied from 200 to 15,000 sec. ft., the average flow being about 450 sec. ft. During floods of 8,000 sec. ft. or more, which seldom last more than 18 hours, the river carries considerable heavy timber and other debris.

Before making any decision as to the type of structure best suitable to dam the river, due consideration was given (1) to a concrete dam of the arch type; (2) a reinforced concrete buttressed dam; (3) a gravity masonry dam.

On account of the uncertainty as to the behavior of the strata at the upper part of the dam, when subjected to horizontal thrust, the first project was eliminated, and owing to the unusually favorable location of the quarry, eliminating costly transportation and handling charges, together with good foundations available for a gravity dam, the cost of a reinforced concrete buttressed dam, even without downstream apron, showed that a structure of the gravity type would undoubtedly be more economical.

The design selected is illustrated by Figs. 1, 2 and 3. The dam carries a highway bridge with 12-ft. roadway, made up of plate girders, embedded in concrete, designed to carry, in addition to the dead load, a live load of 100 lbs. per square foot, and the reinforced concrete floor system is designed to resist the stresses caused by a concentrated wheel load of 8 tons.

The spillway and the two 5-ft. sluiceways have a total discharge capacity of 20,000 sec. ft. The crest is equipped with six automatic flashboards 28 ft. long and 6 ft. high, hinged at the dam crest. Each flashboard is provided with a heavy cylindrical reinforced concrete counterweight located above water, and rolling on a curved track at the bridge piers. These flashboards are designed to begin to drop when the pond level rises 3 ins. above normal elevation, and continue the movement when the river flow increases until they lie flat on the dam's crest. They will return to normal position as the flood subsides, always maintaining the water level at nearly the same elevation throughout their capacity. There are in addition to these four fixed flashboards of the same dimensions, designed to give way when the reservoir is 1 ft. above their top.

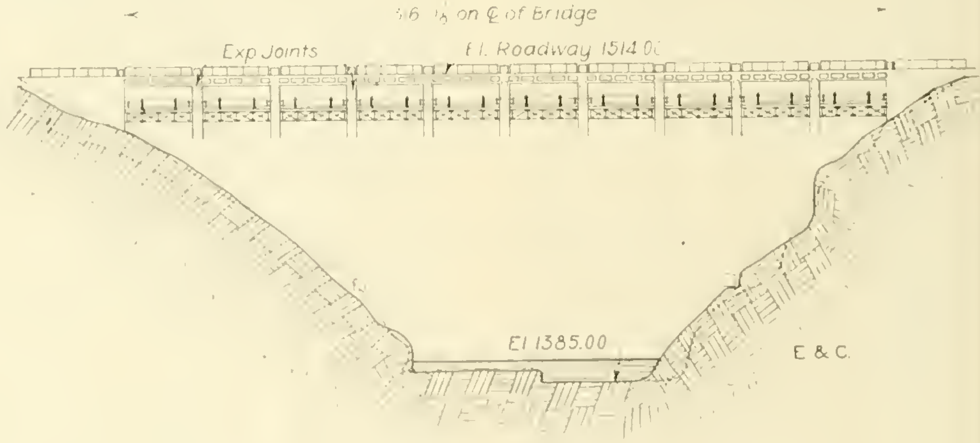


Fig. 2. Front Elevation and Foundation Profile of Concrete Intake Dam, Tallulah Falls Development.

The dam is designed as a gravity structure, its curvature of 900 ft. being disregarded entirely in all calculations of stability. No provision is made for upward hydrostatic pressure, but, to offset this possibility, the weight of the bridge, piers, flashboards and flashboard counterweights, are not taken into account, in the consideration of the stability of the structure against overturning, sliding or shearing. With the reservoir at maximum elevation, there is no tension in the upstream side of the dam, and the maximum compression at the toe does not exceed 12 tons per square foot.

The curvature of the dam was made such as to avoid the use of curved concrete forms during construction, and offsets of 6 ins. were provided every 10 ft. at the upstream side to provide a seat for the forms, which were 1 ft. wide, 12 ft. long, and made up of 4x6-in. studs and 2x10-in. lagging. They were held in place by ⅝-in. round rods hooked up in holes drilled in the "plums." The specifications call for "plums" or stones as large as could be handled by derricks, to be embedded in 1:3:5 concrete, the "plums" to be spaced not less than 6 ins. apart, and be kept 12 ins. from any face of the structure. The percentage of plums is to be at least 30 per cent of the mass. During construction, estimates showed that in the lower part of the dam this percentage was about 36, and the average of the entire structure 34 per cent. Before being laid the plums were thoroughly cleaned of earth and loose materials and washed; they were then laid over a bed of about 12 ins. of very wet concrete. Concrete was then poured and well spaded around and between the plums, and brought up to within 1 ft. of their tops, so as to provide adequate keying for the next layer of masonry. Before laying new concrete, the concrete laid previously was picked and washed under a water jet.

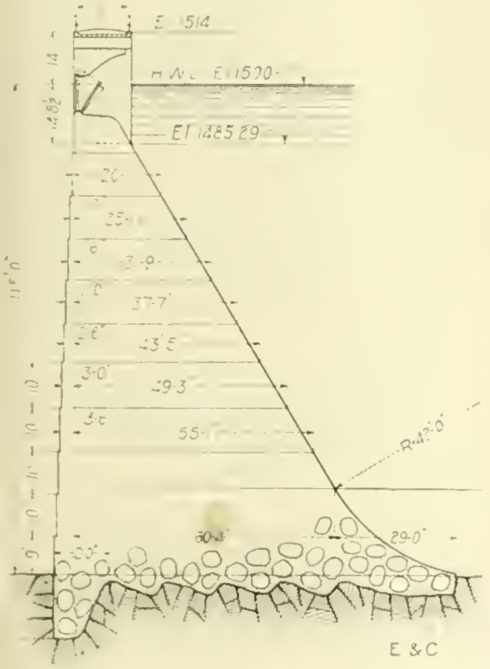


Fig. 3. Typical Section of Concrete Intake Dam, Tallulah Falls Development.

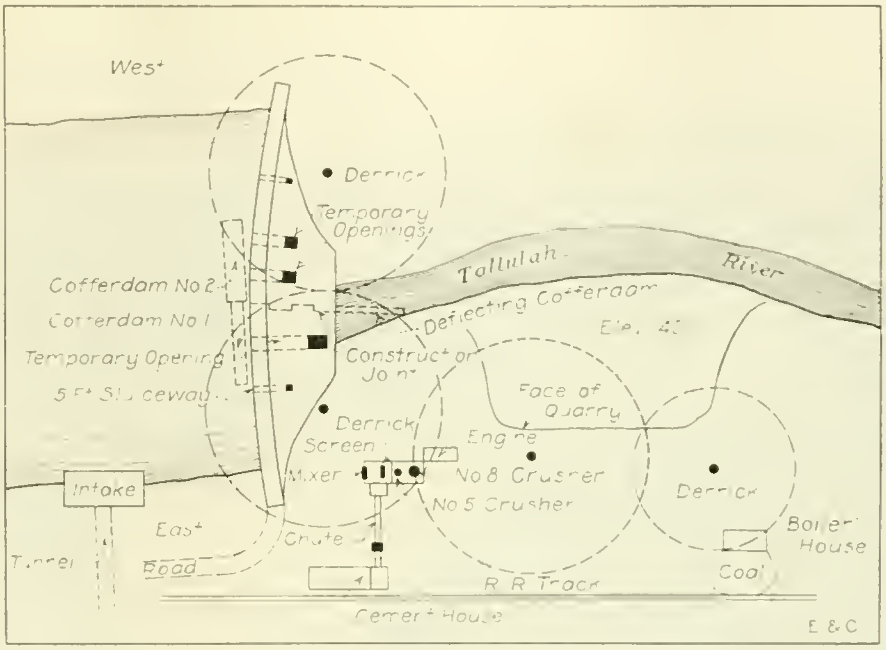


Fig. 5. Plan of Construction Plant Layout Concrete Intake Dam, Tallulah Falls Development.

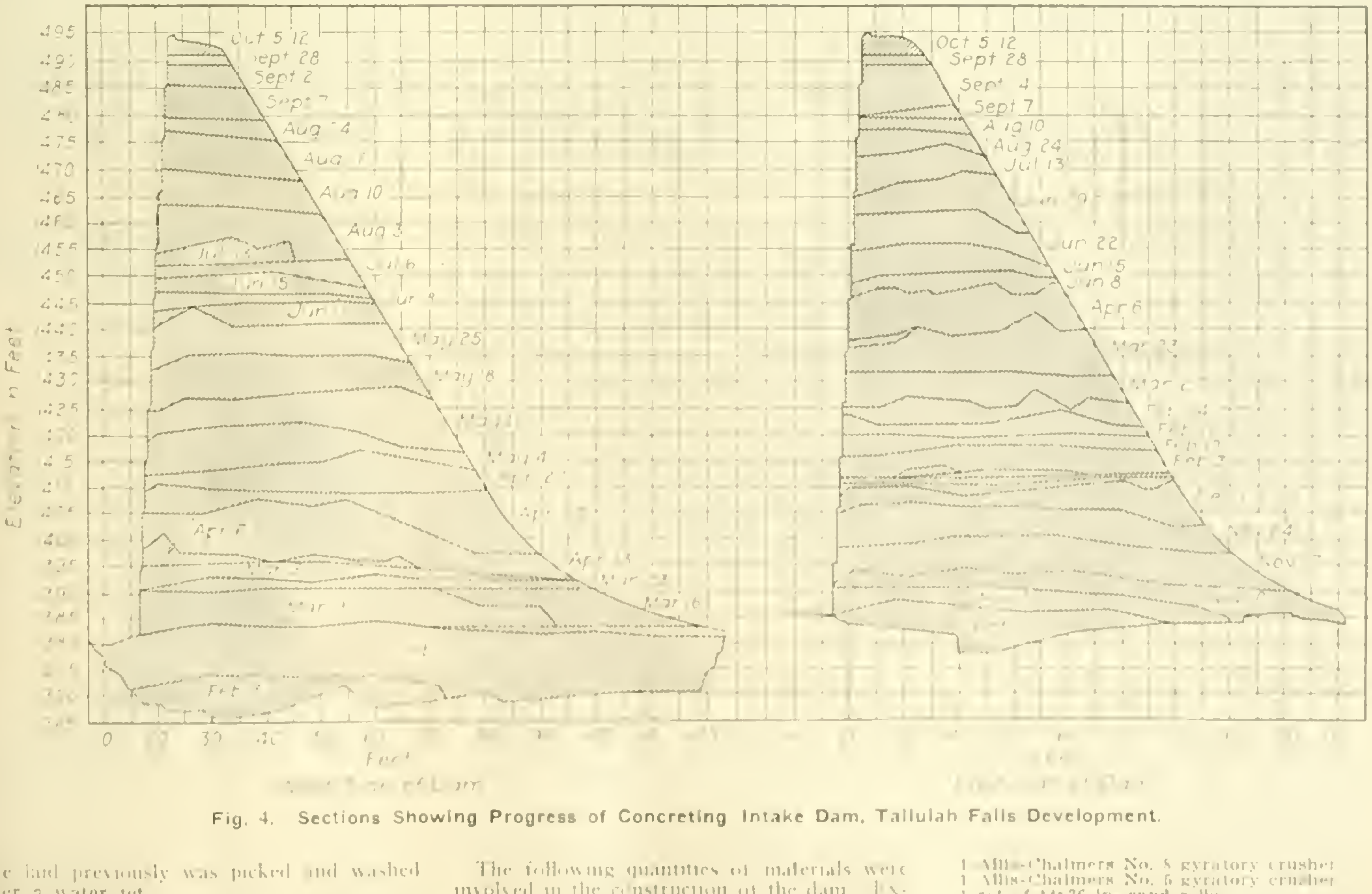


Fig. 4. Sections Showing Progress of Concreting Intake Dam, Tallulah Falls Development.

A quarrying site was available 200 ft. downstream from the dam, thus necessitating no transportation except handling with derricks. No sand being available, crusher product was used, screened so as to reject all particles over ¼ in. in diameter. Royal Portland cement was sold the sub-contractor by the general contractor, at the railroad siding for the price of $1.80, less 10 cts. for sacks, and he was also furnished free of cost 1,000 cu. ft. of air per minute at 100 lbs. pressure, as well as electric current for lighting purposes. The general contractor laid a standard gage track from the Tallulah Falls railway main line to the dam site, a distance of 1,800 ft.

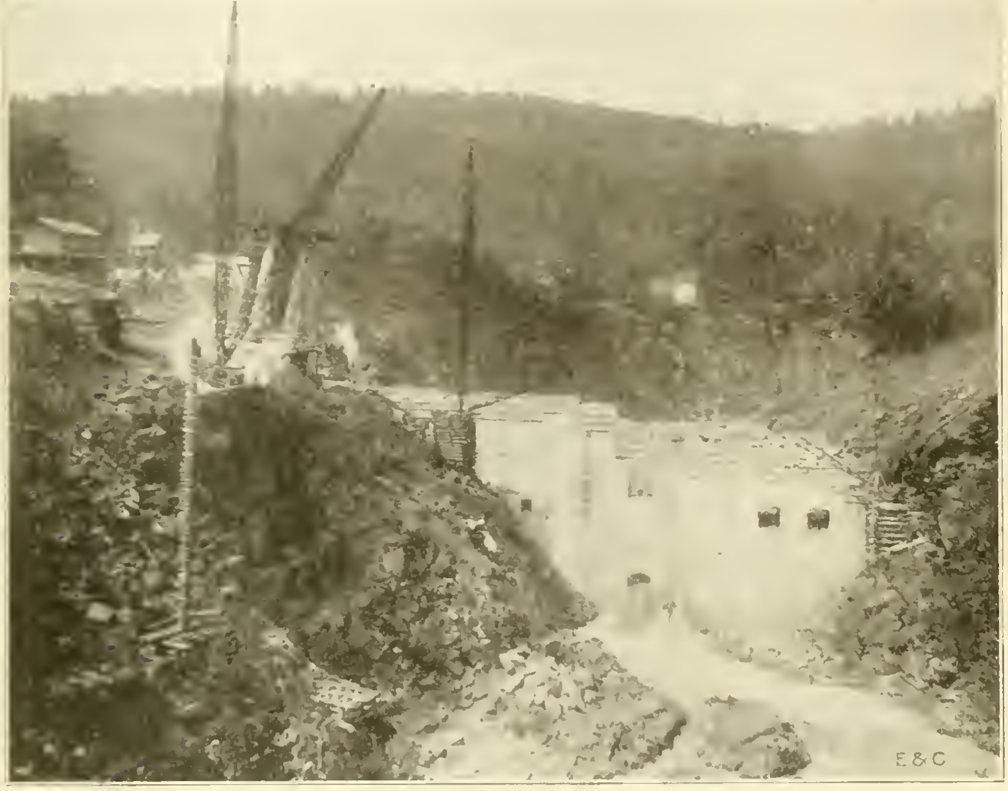
The following quantities of materials were involved in the construction of the dam: Excavation, 10,600 cu. yds., Cyclopean masonry, 39,200 cu. yds. The contract price for excavation, including stripping, earth excavation, rock excavation wet or dry, was $1.50 per cubic yard. The contract price, for cyclopean masonry, concrete in the bridge piers, and abutments, was $1.80 per cubic yard. The setting of gates and steel girders, and the reinforced concrete was paid for separately as extra work.

The construction plant was laid out as shown by Fig. 4, and consisted of the following equipment:

1 200-HP. Hardie Tynes corliss engine.
1 150-HP. and 2 80-HP. boilers.
1 Allis-Chalmers No. 8 gyratory crusher
1 Allis-Chalmers No. 5 gyratory crusher
1 set of 14x36-in. sand rolls
2 American Hoist & Derrick Co. derricks, 115-ft. mast, 100-ft. boom, 15-ton capacity, 35-HP. steam engine
1 wooden derrick, 65-ft. mast, 70-ft. boom, 18-HP. engine

18 7x2x[illegible]-ft. steel skips
3 2½-cu. yd. concrete buckets
1 2-cu. yd. Austin cube-mixer
1 Duplex steam pump.

Excavation at the bottom of the east half of the dam was started in August, 1911, cofferdam No. 1 having been previously built for the purpose of diverting the river to the west half of the channel. The excavation in the east half averaged 5 ft. in depth, the maximum being 10 ft. below the original river bed. Solid rock of extremely hard character was encountered throughout, thus involving little trouble in securing adequate and very secure foundations. The first concrete was laid during October, 1911, and the east half of the dam was carried up about 10 ft. above the foundations. Cofferdam No. 2 was then thrown across the west half of the channel, one end being supported by the finished section, the other by the side of the gorge, the river being then diverted through an opening or tunnel 9x10 ft., provided in the east half of the then completed part of the dam. A deflecting cofferdam was built downstream below the dam parallel to the channel, to prevent water from filling the excavation at the west side, the foundation excavation being carried some 20 ft. below the river bed. Concrete was laid on this side of the structure, beginning February, 1912. In March, a flood of 12,000 sec. ft. took place, and, the opening in the east half of the dam being made inadequate to handle the river, water rose, passing over cofferdam No. 2, then about 30 ft. high, taking away some of the forms at the toe of the west half and about 250 cu. yds. of new masonry. Little damage was done otherwise, and work proceeded without any serious accident or delay.



E & C

Fig. 6. View of Intake Dam Under Construction, Tallulah Falls Development.

(Downstream side of dam showing lower and upper temporary waterways. Also standard concreting forms. On the left hand side, the cement house, crusher plant, concrete mixer and quarry.)

From elevation 1475 to the crest, the structure was carried up throughout at the same level. In order to prevent cracks from developing at the crest, 12 lines of steel rails were laid longitudinally, 6 ins. from the face and 1 ft. on centers over and along the crest and down 20 ft. on the up and downstream sides of the structure.

The lower opening was closed by dropping a heavy timber bulkhead in front of the opening, and a 4-in. drain pipe was provided at the bottom of the tunnel, thus enabling it to be filled in with concrete without annoyance from leakage of the bulkhead. A pocket left above the opening enabled the voids left at the top of the tunnel to be grouted under 100 lbs. pressure. Pending completion of the dam, water was then diverted through two 9x10-ft. openings, 40 ft. above the first opening. These proved very serviceable, when in March, 1913, a flood of 15,000 sec. ft. (largest flood recorded) caused the pond level to rise to elevation, 1,459, i. e., to within 2 ft. from the sill of the intake gates, at a time when both intake and tunnel were still under construction. These openings were closed in a manner similar to that used in closing the lower opening. On Aug. 11 the two 5-ft. sluice gates were closed and water reached the dam crest on the next day. During construction, the force at the quarry, consisted of 50 men and 2 foremen. There was 1 foreman and 10 men at the mixer. Placing of cyclopean masonry necessitated 1 foreman, 3 derrick men and 6 concrete men. After the standard wooden forms had been made so as to be used over and over again, placing and removing of forms during construction required a force of 9 carpenters. Ten hours were worked each day, except Sunday. Four men worked in the blacksmith and machine shops, and there was one timekeeper and one superintendent. The engineering force of the Northern Contracting Co., the general contractor, did all inspection and instrument work, and tested the cement. The following wages prevailed:

Foremen	$4.50 to $5.00
Derrickmen	4.00
Carpenters	3.50
Concrete men	1.75
Common labor	1.50

Two derricks handled the rock from the quarry to the crusher, and two derricks placed the cyclopean masonry, and handled the forms. The two derricks were used also to handle the bridge girders, flashboards, sluice valves, etc. The rock was quarried at a cost of about $1 per cubic yard. Figure 5 shows the weekly progress of laying cyclopean masonry, the average progress being about 1,000 cu. yds.

During construction, the temperature at the dam site varied from 10° F. to 120° F.; yet only one temperature crack developed in the third bay at the east end of the dam, extending about 20 ft. down from the crest. Some leakage was noticable in the lower 3 ft. of the crack, but a few shovels full of ashes and cinders dropped on the upstream side of the dam stopped the leak in a few hours. Immediately after water had reached the crest, i. e., at a time when the masonry had not yet taken the temperature of the water (average 70° F.) the horizontal downstream displacement of the arch crown due to the thrust, at the crest, was measured and found to be ⅛-in. Measurements taken two months later, when the structure had cooled, showed a deflection of ¼ in.

The Tallulah Falls hydro-electric development, including storage dams, power plant, transmission lines and sub-stations, is being built by the Northern Contracting Co. Mr. H. M. Atkinson, president and chairman; Mr. G. W. Brine, vice-president and general manager, for the Georgia Railway & Power Co., Mr. P. S. Arkwright, president; Chas. O. Lenz, chief engineer.

The Hardaway Contracting Co. of Columbus, Ga., was the sub-contractor for this dam, their work at Tallulah Falls being done under the immediate supervision of Mr. R. L. Pearson, their superintendent.

Extension of Transmission System of Nevada-California Power Co.—The Nevada-California Power Co. expects to begin construction work within the next 60 days on an extension of its transmission system from San Bernardino, the terminus of its 229-mile transmission system, to the Imperial Valley, a distance of 154 miles. The company will transmit at 55,000 volts, 3-phase, 60 cycle, and the total capacity of the line will be approximately 4,000 KW. The line will be of cedar poles and aluminum conductor, the spacing to be 6 ft. triangle. There will be three substations on the way, including the terminal substation at El Centro. At this place will be installed a 2,000 KVA. synchronous condenser for controlling the voltage and power factor of the line.

The line will be used to furnish power to the Holton Power Co., and the Coachella Valley Ice & Electric Co. for general lighting and industrial purposes, including the operation of ice plants, cotton gins, oil mills, etc., with a probability of a large consumption of power for irrigating and pumping purposes. All of the stations will be of the outdoor type, nothing being covered excepting the switchboard and synchronous condenser set. C. O. Poole, Riverside, Cal., is chief engineer.

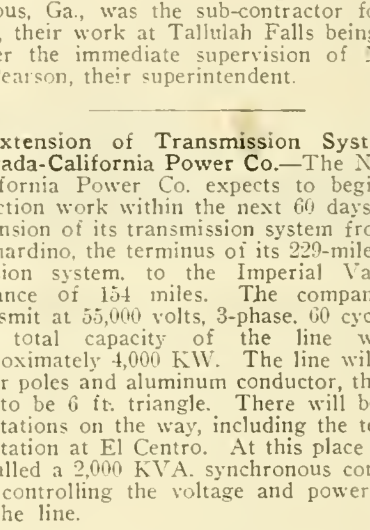


Fig. 7. Details of Flashboard Design, Tallulah Falls Development.

tructural Features of an Interesting Small Hydro-Electric Power Plant on the Sandusky River at Ballville, Ohio.

ontributed by James C. Mills, Saginaw, Mich.

The power development shown by the ac-ompanying plans and described in the article hich follows is chiefly interesting because is of a size and character which fit many pportunities for water power development nd because it teaches the necessity in devel-ping even small water power structures of king full account of flood dangers. The andusky River was one of the rivers of Ohio hich caused disaster in the spring floods of 913. The river has its source in Northern hio and flows for about 115 miles through rich agricultural country, emptying into San-usky Bay. It drains approximately 1,581 uare miles above the power development at remont, O., an area containing only a few nall lakes, consequently the precipitation and esulting run-off collects more or less rapidly the tributaries. Much of the land is of a ch loamy quality underlaid with lime-rock nd shale. As a result of these conditions, f good farm drainage, and of the general at character of the entire water-shed, much i the annual rainfall of about 35 ins. finds s way to the main stream.

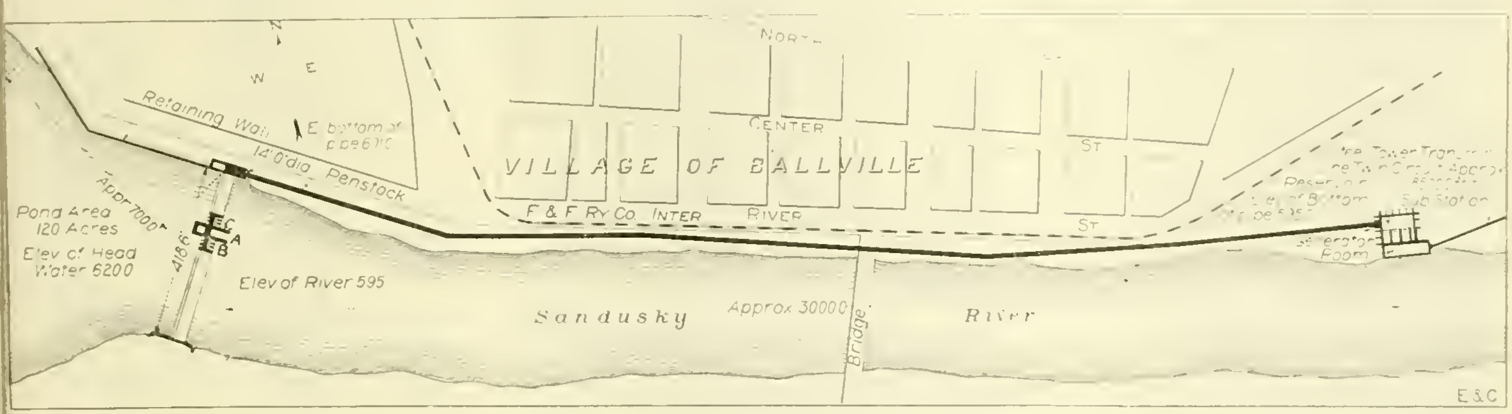
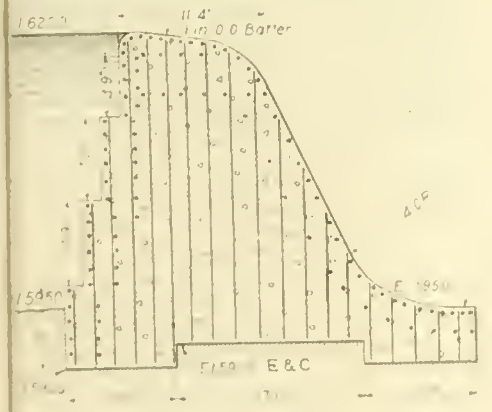


Fig. 1. General Plan of Hydro-Electric Power Plant at Ballville, Ohio.



ig. 2. Section of Dam, Ballville Hydro-Electric Plant.

The work of developing this water power as begun early in 1909, when the Fremont ower & Light Co. was incorporated with a ipital of $1,000,000. In the preliminary stage uch study and investigation was given to evations, dimensions, volume of discharge nder variable conditions, and floods, and nsiderable outlay was incurred in securing e power and water rights. About 115 acres f river land with the flowage rights were irchased, including the sites of three mills d two tanneries. One of these known as e "Old Stone Mill," had ground away for ore than 50 years, taking its power from e old mill race. Meanwhile the market for the electrical energy was being developed, and practically all the "day load" was disposed of to 60 manufacturing concerns in Fremont. Afterward the entire power was contracted for by the Lake Shore Electric, a traction line operating between Cleveland, Sandusky, Norwalk and Toledo.

The power development as first planned comprised a dam and head gates, a steel penstock, a regulating reservoir and power house. The work as finally completed consisted of the same units, but there were material modifications of the impounding unit due to partial destruction of the original structure by the floods of 1913. The finally completed plant is shown in general plan by Fig. 1; this plan helps to explain the original works which will now be briefly described:

ORIGINAL PLANT.

As originally planned and as constructed the dam was 285 ft. long, extending from the south bank to a point which is now about the middle of the river. From this point an earth backed concrete wall or dike extended upstream at an angle of about 45°, and met the north bank in a high hill. From the south end the spillway extended 226 ft. and terminated at the head works *A*, which were 59 ft. long over all. The dam had a base 37 ft. wide, up and down stream, and was embedded in solid rock 5 ft. below the bed of the river. The top of the spillway was 25 ft. and the top of the head works 36 ft. above the bed. The wasteways *B* were provided with three flood gates of white oak construction with ballbearing stands and hoists, and the forebay with a head gate 17 ft. square made of steel and equipped with the necessary apparatus for hoisting and lowering it with one-man power. Trash racks made of ¼x2½-in. flat steel bars were placed in front of the forebay and supported by steel I-beams. Around the entire top of the head works a 2-in. gas pipe railing was erected for the protection of the operator.

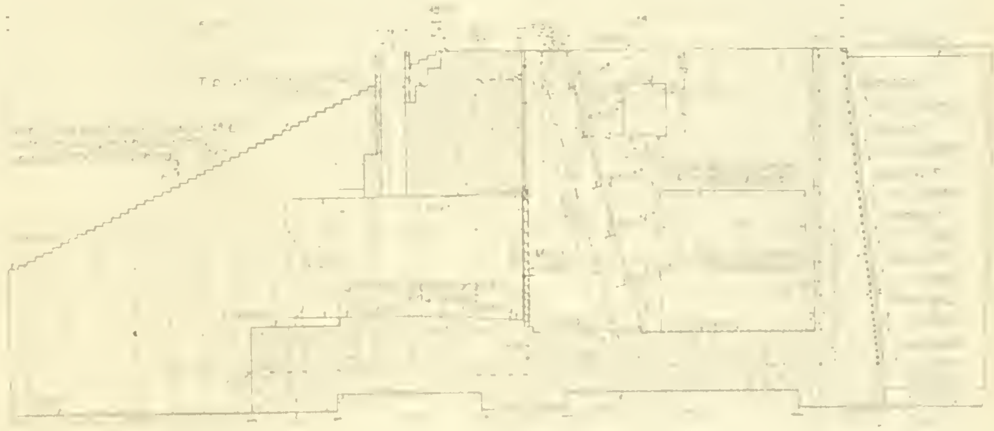


Fig. 3. Head Works with Penstock Connection, Ballville Hydro-Electric Plant.

Into the head works on the downstream side was built the upper end of the pressure pipe line, to convey the impounded water to the power house. This penstock was 3,000 ft. long, 14 ft. in diameter, and was made of ¼-in. steel plates riveted together and provided with an angle stiffener ring at each joint. The pipe was supported throughout its entire length on cement-concrete piers, placed 6 ft. between centers, and laid true to grade with the top not less than 24 ins. above ground. Five expansion joints were put into the line to take care of temperature stress; and three man holes and two washout valves were placed at proper locations.

The lower section of the pipe line terminated in a concrete reservoir having a capacity of about 1,000,000 gals. and adjoining the power house on the north. The walls of the reservoir were 39 ft. high and were heavily buttressed on the outside to withstand great pressure from within. On the south side of the reservoir and built into its walls were four wheel pits, within three of which were installed turbines of 2,000 HP. each, and within the fourth a small turbine of 200 H. P. The dividing wall between the reservoir and wheel pits was pierced with suitable openings into each pit, and provided with steel gates which were operated from the top of the wall by mechanism geared to one man power. In the bottom of the pits were discharge openings with traps for the purpose of cleaning and washing out. The discharge ends of the wheel pits formed the wall of the power house, which was very heavy and thoroughly waterproofed.

The work of repairing the damage and strengthening the pipe line by inserting strong radial bars was well under way, when late in March, 1913, the great floods which devastated Ohio spent a part of their force on the Sandusky River. The concrete wall (before referred to) which connected the head works of the dam with the north bank consisted of an earth fill with a protection wall on the water side. This concrete wall was intended, and was designed and constructed only as a protection to the earth fill in the rear. When the flood reached the dam the spillway, flood gates, and open forebay were insufficient to pass the torrent, and the water gradually rose until it overflowed the concrete protection wall. The earth filling behind it was soon washed away, and the thin facing wall stood alone to hold back a torrent which flowed 1½ ft. over its crest, and 13 ft. over the spillway. It was not constructed to withstand such pressure and on March 25 collapsed, at a time when the total discharge of the river, it is estimated, was 43,000 cu. ft. per second. Of this volume about 4,500 cu. ft. were flowing over the facing wall.

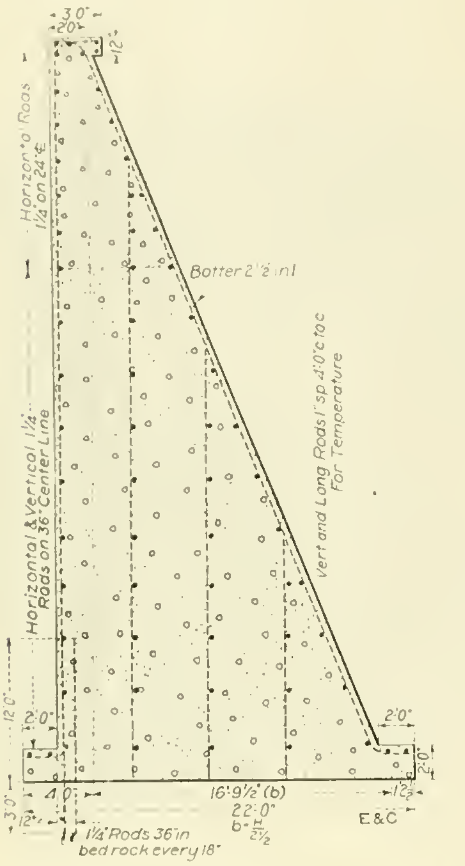

3'0"
2'0"
12"
Horizontal 1¼" Rods on 24" ℄
Horizontal & Vertical 1¼" Rods on 36" Center Line
Batter 2½" in 1
Vert and Long Rods 1" sp 4'0" c to c For Temperature
12'0"
2'0"
2'0"
2'0"
3'0"
4'0"
12"
16'9½" (b)
22'0"
b-H/2½
12"
E&C
1¼" Rods 36" in bed rock every 18"

Fig. 4. Section of Bank Protection Wall, Ballville Hydro-Electric Plant.

When the retaining wall failed the released wall of water tore the reconstructed portion of the pipe line from its foundation piers, and carried it down the river. At the covered wagon bridge a quarter of a mile below a section was carried against a masonry pier with such force as to be wound completely around it. The flood damage to the power project was considerable, and resulted in numerous changes in the original construction plans.

RECONSTRUCTED PLANT.

Figure 1 shows the reconstructed works plan; the changes made were confined wholly to the dam structures and the upper end of the steel pipe line. The earth embankment with concrete facing wall was abandoned and the dam proper was extended 160 ft. north and into the new bank. Referring to Fig. 1 the original dam structure ended at the head works structure *A*.

Immediately adjoining the old head works a new wasteway *C*, 32 ft long and with the same elevation as the other, was built providing for three additional gates 6x9 ft. The old head works were converted into a wasteway, and the old 14-ft. opening intended for the pipe connection was transformed into a gate having a capacity of three 6x9-ft. gates. The total wasteway capacity of the entire work was thus increased to nine 6x9-ft. gates, and the total area from the 162 sq. ft. of the original plans to 486 sq. ft. From the new wasteway another spillway, 103 ft. long, having a crest 25 ft. above the bed of the river, was constructed toward the north. At the north end a new head works *D* containing the forebay for the power connection was built. It is 25 ft. long and 37 ft. high above the river bed. All the new work, like the original, was embedded in the solid rock 5 ft. below the bed of the river. The new dam is thus 445 ft. long, 329 ft. of which is spillway, 91 ft. wasteway for flood gates, and 25 ft. the new head works.

The spillway sections of both original and new dam were alike and are shown by Fig. 2. The body concrete was a 1:2½:5 mixture faced on the curb and downstream face with a 2-in. layer of 1:2 mixture troweled to a sidewalk finish. Figure 3 shows the new head wroks *D*; the concrete was of the same mixture as that used in the spillway and is also reinforced as indicated by the drawing.

Commencing at the north end of the 160-ft. extension a retaining wall was constructed westerly and at right angles with the main work. This wall was designed as a reinforced gravity section having an addition to the weight of the material in the wall itself, 1¼-in. round steel bars anchored 36 ins. into the bed rock and extending up through the wall to resist and overcome the water pressure from the river side. The steel bars were placed every 18 ins. of wall and 1 ft. from the vertical or water face. Steel bars were also placed longitudinally in the wall to take tension and temperature stresses. The section of the wall is 2 ft. on top and 10 ft. on the bottom for a height of 20 ft., with an increasing width on the bottom of 1 ft. horizontal to every 2½ ft. vertical. The footing for the wall is 2 ft. deep and 3 2/10 ft. wider than the base of the wall, there being a 2-ft. heel on the water side. Figure 4 is a typical section, and in connection it is interesting to note the section of the original wall backed with earth which failed in the flood of 1913. This wall section is given by Fig. 5.

The factor of safety against overturning of the new wall is calculated at 2, which is about equal to that of a gravity wall without reinforcement, 2 ft. on top, 12 ft. wide on the base and 20 ft. high, in which the resultant pressure would fall just at the outer edge of the middle third of the base. Without the reinforcement the new wall would have a safety factor against overturning of 1.42. Its stability, however, depends very largely upon the adhesion of the concrete to the 1¼-in. steel bars and upon the anchorage obtained into the bed rock.

The regulating reservoir construction of the new plant is unchanged from that of the original plant and is shown by the drawings of Figs. 6 and 7. The reservoir is rectangular in plan with the penstock entering at one end and the power house abutting on one side as indicated by both Fig. 1 and Fig. 6. Figure 7 shows the wall sections and reinforcement. The floor reinforcement is ¾-in. rods forming a grate of 24 in. mesh. The reservoir is connected by gates with three turbine chambers and the main shafts of the turbines pass through the solid wall of the power house, and are coupled direct to the alternating current generators. These are equipped with governors to regulate the speed of the turbines, and with the switchboards, transformer, and other apparatus make a complete installation. The power house is of concrete and brick construction placed at an elevation above flood waters.

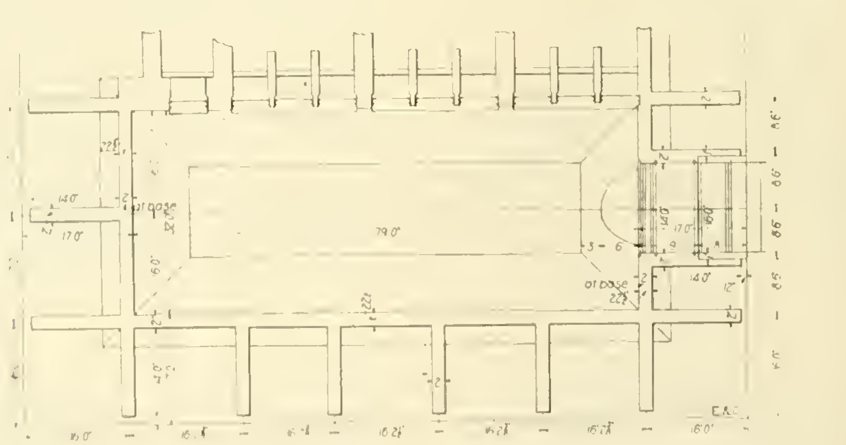


Fig. 6. Plan of Regulating Reservoir, Ballville Hydro-Electric Plant.

Transmission lines comprising about two miles 3-phase distribution connect the power house with the main load cables of the traction line in Fremont. The wires are suspended on steel towers of approved design, and constructed in accordance with the state laws which refer principally to insulation. The size of the wire is calculated to conform with the distance and loads, and is such that the loss does not exceed 5 per cent. A pole line connects the power house with the dam, which carries one set of lighting and one set of telephone wires. The head works are brilliantly lighted, an arc lamp aiding the men in cleaning the trash racks; a telephone booth on the top affords constant communication with the power house.

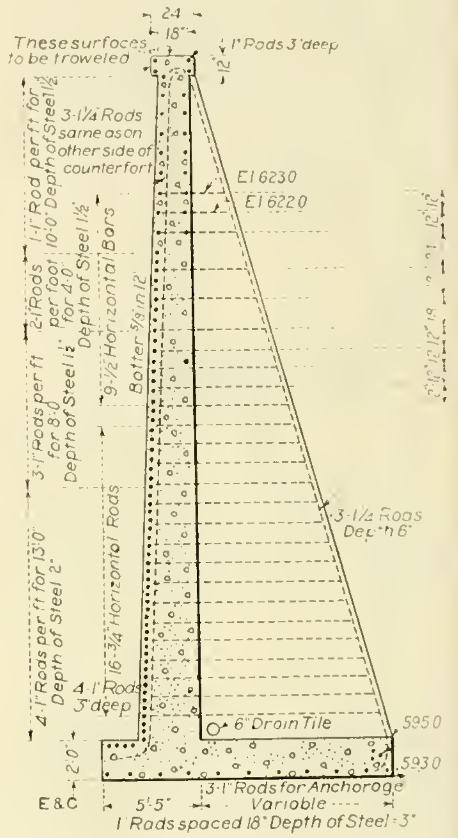
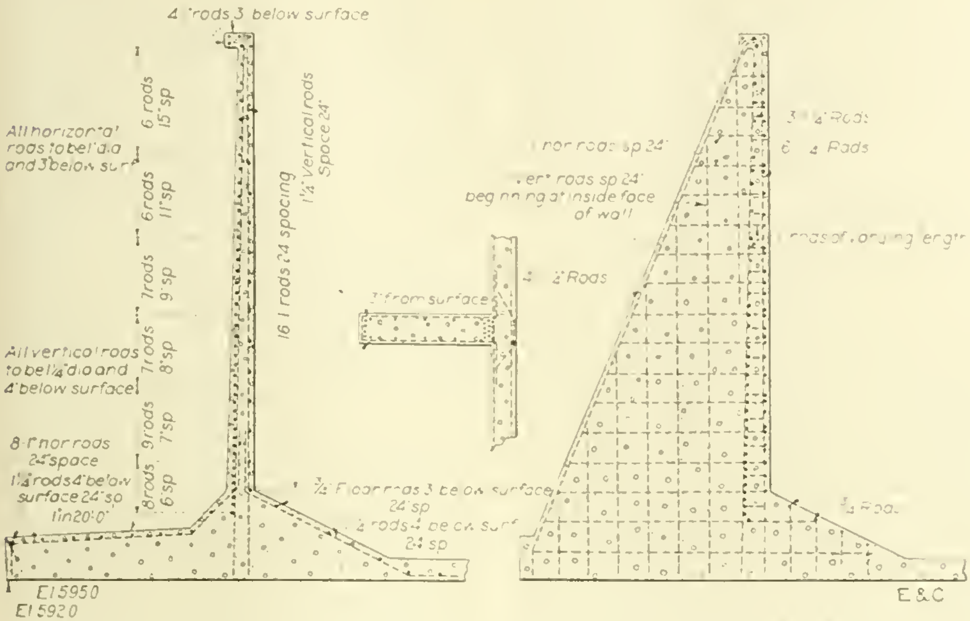

2'4"
18"
These surfaces to be troweled
1" Rods 3' deep
12"
3-1¼ Rods same as on other side of counterfort
1-1" Rod per ft for 10'0" Depth of Steel 1½"
2-1" Rods per foot for 4'0" Depth of Steel 1½"
3-1" Rods per ft for 8'0" Depth of Steel 1½"
4-1" Rods per ft for 13'0" Depth of Steel 2"
9-½" Horizontal Bars
16-¾" Horizontal Rods
Batter ⅝ in 12
El 623.0
El 622.0
3-1¼ Rods Depth 6"
4-1" Rods 3' deep
6" Drain Tile
595.0
593.0
2'0"
E&C
5'5"
Variable
3-1" Rods for Anchorage
1" Rods spaced 18" Depth of Steel 3"

Fig. 5. Earth Backed Dike Wall Destroyed by Flood, Ballville Hydro-Electric Plant.

The engineers for the development plant were W. H. Schott Co., Chicago, Ill. The contractors for the steel penstock were the



Detail of Reinforcement in Wall Panels

Detail of Reinforcement in Pilaster

Fig. 7. Wall Details for Regulating Reservoir Ballville Hydro-Electric Plant.

Riter-Conley Manufacturing Co., Pittsburgh, Pa.; the contractors for the electrical equipment were the Westinghouse Electric & Manufacturing Co., Pittsburgh, and the contractors for the water-wheels were the James Leffel Co., Springfield, O.

Large Engineering Project in India.—Work is under way on the construction of the dam across the Musi river, near Hyderabad in southern India, with the object of preventing the disastrous floods to which that vicinity is subject and of providing a supply of pure drinking water to the two cities of Hyderabad and Secunderabad. This dam will be 120 ft. high and will afford facilities for disposal of flood waters so that they will not come through the city of Hyderabad. The drainage area intercepted will be 250 sq. miles, and about 6,500,000,000 cu. ft. of water will be stored for irrigation and municipal purposes. There will also be an average area of about 335,000,000 sq. ft. to a depth of 9 ft. for moderating the floods. It is proposed eventually to construct two storage reservoirs for storing the excess discharge in a flood, one on each branch of the river, 6½ and 8½ miles, respectively, above the city. The capacity of the East reservoir for storing flood water will be 8,370,000,000 cu. ft., and that of the Musi reservoir 5,360,000,000 cu. ft. This equivalent to a runoff of nearly 6.9 ins. from 862 sq. miles, the entire catchment above Hyderabad. This work is under the general charge of the Public Works Departments, and under the immediate direction of an experienced European engineer, Mr. A. T. Mackenzie.

No approximate figures have been furnished as to the probable cost of this undertaking, but the various works connected with constructing a dam 120 ft. across a river draining nearly 1,000 sq. miles of the country and subject to violent floods, also constructing conduits 12 miles long and a pipe system for both drinking and irrigation purposes, will require expenditures of very large amounts.

Western road enthusiasts are proposing a highway for automobiles connecting the Pacific Coast with Dawson City, Y. T. Automobiles have already made the trip from Vancouver to Hazelton, in the northern part of British Columbia, and with difficulty from Whitehorse to Dawson.

Water Works

The Consulting Engineer in Water Works Practice.

Contributed by John W. Alvord, Consulting Engineer, Chicago.

The fundamental principle of human association is that unless you give valuable service for your time and your generation, your generation promptly forgets and ignores you. If the consulting engineer exists at all it is because he is found to be necessary in human society as now constituted.

The popular idea of a consulting engineer is that he is a human wizard who sits in a well-furnished office and imparts large chunks of specialized knowledge, off-hand, and without any particular preparation, in reply to any question that can be asked him in his specialty, all for a fabulous fee, received in advance if his guests are unknown to him.

This is delightfully untrue; the real fact is, that a successful consulting engineer is a man who, after many years of ordinary and salaried experience in a given line of engineering work, gradually fits himself to pursue the broader aspects of that line of work; accumulates a large amount of general data, a specialized library, and an experience with a large variety of conditions in his chosen field so that he is enabled to study certain problems more efficiently than most people engaged in the ordinary way in that line of work.

Far from being able to answer any kind of a question off-hand, the consulting engineer is usually noted for his tendency to answer nothing whatever without opportunity for the compilation of the necessary facts and the time for a studious and judicial interpretation of their meaning.

The ability to do this kind of work effectively means generally many years of training, the close study of relatively few subjects as well as the accumulation of special data and indexed information known among the initiated, colloquially, as "dope."

A good many engineers would like to become consulting engineers but do not have the grit and persistence to follow out all the exacting requirements, involving, as they do, the investment of much personal time and means as well as the sacrifice of much social and home life. Let us see in brief what some of these requirements are:

In the first place, the consulting engineer must have an investigating type of mind. That is, a type of mind that can accumulate evidence, sum up facts, and procure information without coming to premature decisions. This is not the common type of mind, and usually has to be cultivated. Most of us naturally jump to conclusions, and the less we know about a subject often the more hastily we conclude on something about it.

In the second place, the consulting engineer must subordinate the desire for personal gain to the pursuit of truth for its own sake—he must be willing to face a personal financial loss if he should become inadvertently so involved that it is necessary, to do so, rather than produce inefficient results, or deliver undigested opinion. If he undertakes a problem at all, he will be expected to serve his client's interest in the light of truth and fact even if that involves a personal contest with his own client and a financial loss to himself. In other words, no man who has not a high sense of honor, a keen appreciation of principles for their own sake, and a disregard for emolument, when emolument interferes with principles, can hope to succeed, or at least succeed very long as a consulting engineer.

In the third place, a consulting engineer should have a broad experience behind him upon which to base his work. The investing public instinctively distrusts youth. The old saw, "Young men for war, old men for counsel" does not seem to lose its force as an axiom. The speculator usually demands that the man upon whose judgment he stakes his money shall have arrived at years of discretion, and in the long run this is founded on good common sense. It is a disappointment to the young man who, with his ambition and vigor, would like to leap at once into the final fruits of a long career, but there can be no better and sounder foundation laid for the work of the consulting engineer than 15 or 20 years' experience in the superintendence, construction, and design of engineering works—in subordinate position if need be—with all the experience in human nature, and the opportunities for self-improvement that that length of salaried service implies.

In the fourth place, a consulting engineer must of necessity have some capital and business experience to successfully operate in his calling. To enter the consulting field with insufficient means is not only to be daily harassed by business anxieties, but is often a fatal handicap to the engineering judgment in matters where the engineering judgment should be carefully protected against baneful influences of a financial character. There is probably no more peculiar and sensitive strain upon the morals than is afforded by the financial temptations of an insidious nature that confront the young and inexperienced engineer who has too early embarked on an independent career. The temptation to slight his work when he has named too low a fee—the temptation to accept work from a client who clearly wants to use him improperly, or the temptation to report favorably on a doubtful project because it may lead to more work—in these and a thousand other insidious ways are embarrassments created for the financially helpless, which do not so often come to the old established and well founded practitioner.

Ordinarily it takes a considerable amount of capital to start a consulting engineer practice. The less wide one's acquaintances and the less mature one's experience the more capital it takes to provide for the cost of "Going Value," that is to say the cost of financing one's experience and personality into a paying basis in the engineering field.

Not only does it take capital to start such a practice but it requires a considerable capital to conduct it. While it is a safe business, in that accounts are almost always good and collectible, collections are not always possible until work is done. It is safe to say that the present consulting water works engineers in this country, require a working capital always invested in their business of not less than $10,000 to $25,000, and perhaps at times of even more.

In the fifth place, the consulting engineer must expect to do many things for the public and for the advancement of his profession without any direct remuneration, not that he should do work that can and ought properly to be paid for, but many times it is desirable for him to make original investigations costing sometimes many thousands of dollars, the results of which it is wise for him to give to the public and to his professional brethren. In the matter of information the biblical injunction "freely we have received freely give" is as good an axiom in engineering as it is applied to the religious life. No successful consulting engineer ever made his mark by hoarding expensive information. Very few consulting engineers have patented their ideas extensively and remained consulting engineers. The professional world expects and demands of the consulting engineer that he be generous with his surplus time and means, and they accord honor and reputation only to those who live up to their opportunities in this regard.

In the sixth place, the consulting engineer must have the acquaintance and the long personal confidence of men of affairs who have need for his services from time to time. This is really his first and fundamental requirement but it is necessarily the result of all that has been heretofore outlined.

The work of the consulting engineer is based primarily on the personal conviction of his acquaintances after many years of observation that he can be of real use to them in difficult problems. After years of established reputation, others may come to him on the basis of his reputation alone, but in the start he must depend on this element of personal confidence. The best advertisement a consulting engineer can have is a satisfied client.

In municipal work and municipal water works work particularly, the consulting engineer labors under the handicap that rotation in municipal office often deprives him of influential friends whose confidence he has established by his work for their city in the past.

The consulting engineer is peculiarly dependent for his reputation on the fellow members of his own profession. Nobody knows so well how efficient or deficient an engineer is in any given line, as do his professional brethren. If you want to be sure to get a good doctor, inquire about the one you have in mind among other doctors in his vicinity and you will generally get a line on him. So it is with engineers, to select the right engineer for a particular problem the searcher for talent can do no better than to advise with other engineers in kindred lines. Nobody knows engineers like engineers, and an engineer who stands well with the rest of the engineering profession, and is recommended by them, may usually be safely employed.

So far we have spoken about engineers as a class. The water supplies of our several thousand cities in this country in connection with their sanitary and other municipal problems, afford, perhaps, the largest field for consulting engineering that obtains among engineering specialists. This is because water supply and sanitary problems are sporadic. In railroad work, railroad operative problems are constantly arising and are generally local to the road, consequently railroads have to continuously employ engineers expert in their work. As a result there are few independently practicing consulting railroad engineers available. In bridge and structural work, there are a larger number available but it is safe to say that at least half of the consulting engineers in this country are occupied almost solely with municipal problems, which are many and varied.

The consulting engineer in water works practice has the opportunity to make himself useful in special ways that perhaps should be noticed in some detail.

In the first place, he is, or should be, familiar with all of the more important water supplies of the country, so as to understand in a general way their peculiar working conditions and special characteristics. In addition to a general familiarity with such plants, he should have a detailed knowledge of a large number of plants with which he has personally been identified and from which he has drawn his experience. It is quite as important that he have a good working knowledge of small plants as of large plants, for as a rule, more skill and care is required to adapt means to an end in small water plants than in large ones, besides which, one must realize that only a good perspective can be obtained when one has in mind measures which are applicable to all sizes and varieties of works.

In the second place, he should desirably have close and intimate touch with water works men and interests all over the country. He should have a specialized library of all the water works literature, carefully filed and indexed, for ready reference. He should also have the notes of his own past work so indexed and accessible that they are almost instantly available. And if he be wise he should expend a good deal of time and money in tabulating data forming a data file for ready reference on all of the more independent subjects that are likely to come before him.

A library of special water works reports by other engineers, chemists and experts, should form a most valuable part of his library, and this cannot ordinarily be accumulated without special watchfulness through the professional courtesy of his fellow practitioners, covering a considerable period of time. A well-equipped, modern, up-to-date library in a water works engineer's office represents an investment, with its accompanying files, of from $3,000 to $15,000.

In the third place, if he is to be of the greatest good to the greatest number of clients, he must have a well organized force of assistants, familiar with his methods and ideas, and capable of carrying them out, ofttimes at a distance from the home office. Engineering assistants of the proper type of mind needed by the consulting engineer must be trained in his own office by a well arranged system of precedence, promotion, and personal interest and instruction. With such a force the personal influence of the chief of the office can be spread out over a very large field without making the cost in any one investigation unduly expensive, because in a well organized office no man should have to do what a less expensive man can do equally well, and the chief of the office can confine and limit his time to those things only which are required for direction and general supervision to the financial advantage of his client.

In the fourth place, the water works engineer must continue to be a close student of the art all of his professional life. So soon as he begins to relax his attention in this direction, so soon does he begin to drop back and be in danger of being superseded. No matter how busy he may be, or with what great responsibilities he may be charged, he cannot afford for one instant to relax his study time. New methods must be investigated where they have promise. Water works literature should all pass under his eye, at least to the extent that he catches the meaning and purport of every important water works article, paper, or book that is presented during the current year. The water works conventions in which he is interested must be attended, and he must see to it that he is constantly rubbing up against men and minds and keeping up with the current of thought in his special line of work. The exacting requirements of this kind of study, in connection with the business of operating an office, pursuing special problems, and at times shouldering the responsibilities of construction, is a severe requirement, and with most men compels much evening work, especially in early life. The social sacrifices which most of the prominent water works engineers of this country have undergone would surprise many a young man who saw only the emoluments and the rewards involved.

In the fifth place, the water works engineer in consulting practice should have breadth of view outside and beyond the strict limit of his own profession. His more especial field is, however, a broad one and may be outlined as follows:

In water source inquiries he must be informed in meterology, hydrography, and storage problems as well as all of the specialized work done in the underground waters. In questions of quality, he must have a general working knowledge of chemistry and bacteriology so as to make correct interpretations, weigh relative advantages and disadvantages of different waters, test those in use, guard the community against disease and epidemic and decide on questions of filtration. In mechanics, he must be well acquainted with all of the various machines, appliances, and fuels, and deal with questions of combustion, station economy and transmission problems, all of which go to the successful solution of hydraulic power equipment and economics. In hydraulics, he must, of course, have a wide knowledge of the various properties of water and its control. In law, he must be proficient with the details of municipal methods and checks, and the relations expressed in contracts. As an administrator, he must know how tactifully to manage men, talk to Boards, Councils, Committees, deal with contractors, and superintendents. In economics, he must know how to reason about finance, understand the conservation of capital, relation of time to cost, and the specific economic questions that always accompany matters of public policy. In matters of public health, he must be fully acquainted with the theories of disease, the etiology of epidemics, and the laws and methods of controlling the sanitary surroundings of the modern community. Now all of this means that to the owners of the water supplies of this country there is available on comparatively short notice in a considerable number all of the means and requirements of an organized force, for the rapid study and solution of the many difficult problems that occur in the art of municipal water supply.

Probably the greatest advantage that occurs to the local water supply management from the engagement of the consulting engineer, lies in the fact that to the solution of local problems can be brought a very wide range of comparative study with the possibility of improved methods and the application of a variety of means. This comes only through a wide acquaintance with the successful ways in which difficulties are overcome elsewhere. The more water works officials and managers mingle in the water works conventions or visit varied plants in the different parts of the country, the more they realize the value of this broader aspect of their problems, and appreciate the work that the consulting engineer can do for them.

A second valuable office which the consulting water works engineer can perform for the local water works is to bring to its investigation a mind fresh, unbiased, and free from any previous local tradition or precedent; not that local knowledge is not important and valuable and must always be carefully considered, but ofttimes a review of many possible methods are necessary, and such a review often throws certain methods into strong relief that might otherwise be passed by. No one knows better than the consulting engineer himself that logical research among facts, with a correct and unbiased summation, may often reverse a pre-conceived opinion held at the beginning of the investigation.

A third valuable aid which the consulting engineer can give to the local water works management is to give perspective to the problem in hand by shaping up all the alternatives, so that after a series of projects are complete on comparative basis the management can see as clearly as the consulting engineer why a certain course of improvement is desirable and necessary. Indeed, it should be the special art in all engineering reports to so arrange the data and contrast the opposing courses of action in a series of estimated projects that the desired decision shapes itself almost unconsciously from the facts contained in the report. It is often well in comparative work of this kind to include projects that are not favored for the sake of fairness and to throw into stronger relief a desirable selection.

The consulting engineer therefore, is not a dictator or an autocrat. He is not a wizard or a prophet. He is simply an intelligent human being who has learned how to lay hold of a special problem in a patient and orderly way with the means that are open to any one, and finally dissolve the mists that hang about the difficulty so that the layman can understand the matter as fully as the engineer. All this is accomplished by taking one step at a time, with a reason for every step.

Only the more intelligent of the water works community utilize such services. In a large majority of water works plants the consulting engineer is unknown, or, if known, he is called only rarely to remedy blunders, or a succession of blunders, caused by overconfidence. Particularly is this true in municipal work. The rapid rotation in office of the American municipal official causes him to imagine that he must decide all matters off-hand as best he can, and the idea that many municipal problems are matters for careful and specialized study does not ordinarily appeal to him.

Only as we advance in civilization and better government will we realize that a thought must preceed an act, and the more important the act the more necessary it is to have careful preliminary thought.

Strictly speaking, the consulting engineer is one who gives his time wholly to advance and specialized study. As a practical matter, however, most consulting engineers undertake more or less construction. This is, in the main, desirable as it keeps them in touch with the practical side of their profession and prevents their becoming mere theorists.

Some consulting engineers in water works work limit themselves annually to a few large undertakings in construction, while others will undertake a limited amount of all kinds of construction, both large and small. The latter, of course, develop a larger working organization than the former, and while a greater variety of practical experience results, probably such engineers have less time for the studious side of their profession. Nearly every consulting water works engineer has some specialty to which he has given more than ordinary thought and attention.

The fees which water works engineers receive cause them to be regarded ordinarily with some envy by their salaried brethren, but there is also another side of the picture. The better class of well known consulting engineers ask $100 per day for their time, $50 per day is common, and from $200 to $250 is not unusual, but such men do not expect to do any work that can be just as well done by a competent assistant, so that the amount of time which they ordinarily charge to a problem is quite limited. Nor can they expect to maintain that rate constantly no matter how busy they may be. The business of their office, their correspondence, time taken for special studies, attendance at conventions, and committee work, make large inroads on their annual schedule, and it is safe to say that the water works consulting engineers are one of the most approachable classes of men there are. They do not expect to earn money all the time. The expenses of a consulting engineer are usually quite large. He must be a member of many technical societies and attend their conventions. Often he is called upon to do elaborate and extensive committee work in this connection. In many ways he finds it desirable to make much independent research. It is safe to say that a consulting water works engineer who has offered him all the work he can attend to, will be unable to properly charge up more than two thirds of the calendar working days in the year and will often otherwise expend from one-sixth to one-third more of his time for the benefit of his clients, his profession, and his office efficiency.

The work of the consulting engineer calls for a great deal of arduous travel—from 20,000 to 60,000 miles a year for the busy man. It takes a robust constitution to stand all the changes in climate, winter and summer, with all the varied diet and hotel fare that continual travel involves. The engineer is usually expected to work from 10 to 16 hours a day when away. Often many nights a week must be spent on a sleeping car, and with anywhere from 100 to 200 days a year away from home, life is not always a joy.

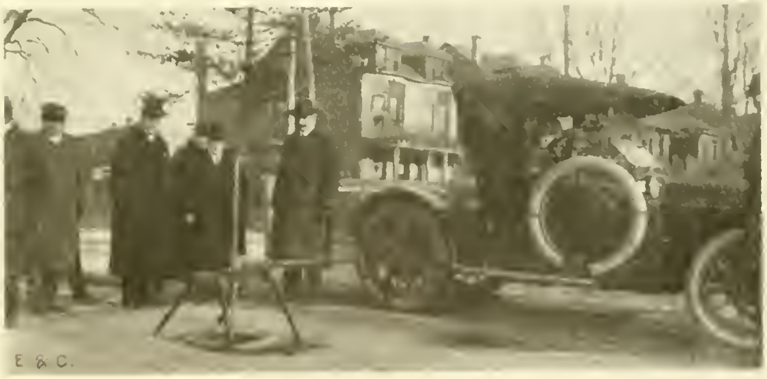
But the work of the consulting engineer unquestionably has its own peculiar rewards, for he is thrown into contact constantly with bright minds. He is constantly confronted with new and interesting problems, and filled with the zest of being needed and useful. Charged with great responsibilities for the public health and welfare, his work is undoubtedly most stimulating and inspiring.

A New Portable Power-Driven Device for the Rapid Opening or Closing of Large Gate Valves.

(Staff Article.)

The accompanying view shows a device recently successfully tried out at Trenton, N. J., for the rapid opening or closing of large gate valves in water mains. The device possesses two important advantages, namely, its use makes possible the dispensing with practically all the manual labor now necessary in the opening and closing of large valves, and performs its function with a rapidity that would be impossible under the old method.

As shown in the illustration the device consists essentially of pieces of horizontal and vertical shafting connected by a bevel gear and provided at the other ends with proper connecting devices for attachment to an automobile wheel and to the stem of the gate valve. The vertical shafting is supported by a bench set up over the top of the valve chamber as shown. Provision for varying depths to valves is made by moving the vertical shafting up or down as required by releasing and subsequently tightening the horizontal gear wheel upon this shafting, the length of this shafting being sufficient to give the necessary range of movement. The horizontal shafting telescopes so that a fixed distance from the automobile to the valve chamber is not necessary. One of the power wheels of the automobile is, of course, chosen for attachment to the horizontal shafting and this wheel is jacked free of the ground; the revolution of the wheel then opens or closes the valve as desired. The device can be adjusted to an automobile wheel of any size.



View of New Portable Power Driven Device for the Rapid Opening or Closing of Large Gate Valves.

The recent demonstration of the device at Trenton was made upon a 12-in. valve. Although the device has not as yet been perfected the demonstration indicated that it will be a success. The valve was closed in 1 minute and 15 seconds, and was then opened in about the same time.

This appliance will have its chief field of usefulness in large cities where there are many large mains and where quick closing of valves is at times highly desirable. The breaking of large mains is by no means uncommon and frequently such breaks cause the flooding of basements where valuable goods are stored, thereby causing much damage. Numerous other property damages are caused by the breaking of large mains. Shutting off the water from a broken main is the first aim of the department when such accidents occur and the prompt reaching and closing of the controlling valve is highly desirable. Since nearly all large water departments are now provided with at least one automobile the device here described seems destined to be of great service in its proper field. It is the invention of Mr. W. H. Van Winkle of the Water Works Equipment Co. of New York City and will be placed on the market by that firm.

Methods Employed in Laying an 8-in. 9-Mile Wood Pipe Line for Railway Water Supply at Stoughton, Canada.

The engineer in charge of the water supply of the Canadian Pacific Railway Co. selected Rock Lake as a source of supply at Stoughton, a division point in the province of Saskatchewan. A pumping plant was installed at the lake and a wood pipe line 8 ins. in diameter and 9 miles long was installed to convey the water to Stoughton. The design of the necessary works and the construction methods employed in laying the pipe line were described in a recent issue of The Canadian Engineer substantially as follows:

The country traversed by the pipe line is clay loam at the surface, while near the lake quicksand is reached at a depth of 6 ft. from the surface. At this depth, on the higher land, a mixture of sand and clay is found.

The plant at the lake consists of a pumphouse and a house for the attendant. A well 16 ft. in diameter and 16 ft. deep was sunk near the shore of the lake. The walls of the well are 8 ins. thick, of reinforced concrete, and a 4 in. slab, of the same material, covers the well. An 8 in. cast iron pipe, with a fall of 1 in 250, conveys water from a point 90 ft. off shore to the well. The pump draws from this wet well and discharges into the wood pipe, which is wire wrapped, pitch-coated and tested for a head of 350 ft. This pipe line was laid at a depth of 8 ft. for protection from frost. In short swales and other places where it was impossible to get a good joint by following the ground line, the pipe was put down as far as possible and extra material was placed over the backfilled trench. Air valves were located on all summits, there being nine of them on the entire line. There are four gate valves at approximately uniform distances, but located in sags wherever possible. There is also a gate valve in the cast iron pipe on the intake side.

The contract for ditching, laying and backfilling was awarded to the J. A. Broley Co., of Fernie, B. C. The contractor started work on the ditch close to Rock Lake and worked toward town. Harvesters' cabooses, drawn by horses, were used as cook and sleeping quarters. An Austin drainage excavator fitted with caterpillars was used for ditching. At the start it was not very successful, owing to the presence of quicksand, which filled in before the pipe could be placed. As a result it was decided to leave all bad places to be finished by hand. When the machine reached higher ground it made better progress, although a great many delays were caused by the bucket chains breaking by being wedged against stones, which were quite numerous in places. As the pipe was so easily laid in the dry places it was often put in during overtime, the men working on the wet spots during the regular day. When the machine was digging in sandy soil which was not too closely packed it could make 800 ft. a day, but sometimes breakdowns to machinery from the above unavoidable causes would reduce this to one-half the amount

The pipe was prepared for driving by filing the spigot end so as to allow the pipe to enter the sleeve easily. This filing also prevented the end of the pipe from splintering, and the fibres from turing back and causing leaks. After filling, a little gear grease was applied to the spigot to aid in driving; the sleeve end of the pipe was also carefully inspected to see that no lumps of pitch adhered to the wood, as this is also a possible source of leakage.

The longest stretch of pipe laid in one day was 1,629 ft. The number of men actually engaged in laying was as follows: One foreman; one man filing and greasing spigots of pipe (part of time and tamping part of time); two men on surface dropping in pipe, sometimes having to carry pipe for short distances; and three men in the ditch driving pipe and removing material left by machine. The men in the ditch also attended to the tamping when held up for pipe or other reasons.

The pipe was driven by a dolly made of a tamarac tie about 5 ft. long and ironed on each end to prevent brooming. It was suspended from a chain attached to a plank across the ditch. The pipe was protected by a plug of fir inserted in the sleeve while driving.

The wet places were dug by hand, sheet piling being used to hold the bank up. Several methods were used by different foremen, but the best results were obtained by digging holes from 8 to 12 ft. long according to the conditions, leaving a space of 2 or 3 ft. between. This space was afterwards tunnelled out. By this method less sheet piling was required and as the pipe was laid, the water was dammed in the tunnel reducing pumping to a great extent; generally an ordinary diaphragm pump was all that was necessary—although larger pumps had to be used in several cases.

Where sloughs with water on the surface were encountered the method of procedure was to build a puddle dam about 6 ft. on each side of the center line, and then to pump out the space between, by gasoline pumps. In one case a 6 in centrifugal pump was necessary as the quantity of water was too great to be removed by slower methods. After the water was removed from the surface the method of excavation above described was suitable.

Some difficulty was experienced during wet weather on account of the pipe being raised by the bouyant force of the water which filled the trench at low points. In some cases the pipe dropped to place on the removal of the water but where sand had worked in under the pipe it was necessary to remove the pipe, clean out the ditch and relay the pipe. This trouble, however, would not have occurred if the backfilling had been done regularly and kept close to the pipe laying, the work being in the hands of a sub-contractor. A mormon scraper was used for backfilling the trench, one team and two men being able to do 700 ft. of dry work or 200 to 300 ft. of wet backfilling per day.

Economic Aspects, Methods and Results of the Mechanical Cleaning of Water Mains at Hartford, Conn.

An unusual amount of specific information on the economic aspects, methods and results of the mechanical cleaning of water mains, was given in a recent paper by Mr. Caleb M. Saville, Chief Engineer of the Board of Water Commissioners of Hartford, Conn., before the New England Water Works Association. The information here given relates to the cleaning of certain main supply lines of Hartford during 1912 and is from Mr. Saville's paper. Especially interesting and timely comment is made in the paper upon subsequent corrosion of cleaned mains and related subjects, such as increase in red water troubles. The data given are very favorable to mechanical cleaning.

The city of Hartford, Conn., is supplied with water by gravity from a system of six reservoirs in West Hartford, distant about 6½ miles from Hartford City Hall. Nearly 12 miles of watershed are tributary to this system, the drainage from all of which finally flows to Reservoir No. I. The effluent gatehouse is located at the base of the dam of this reservoir, and from it lead the three mains which supply the city, two 20 ins., and one 30 ins. in diameter. The first main laid was cement-lined, and was put into service in 1867. On account of frequent breaks, portions were replaced from time to time with cast iron until now this is wholly a cast iron line. Although the older in point of original location, this main is actually the lesser in age of the two 20-in. mains. In speaking of this main hereafter, it will be known as the south 20-in. main. The other 20-in. main, known hereafter as the north 20-in., was laid in 1875, and was, therefore, about 37 years old when cleaned. The 30-in. main was laid in 1896 and was 16 years old when cleaned.

The average consumption for the year 1911 was at a rate of 8,051,000 gals. per day. The maximum monthly rate in 1912 was 9,245,000 gals. per day, the maximum rate at any period during the day being about 13,700,000 gals., while the minimum flow from midnight to 5 a. m. was at the rate of 4,500,000 gals. per day. The drop in pressure at the City Hall due to this difference in rate was 18 lbs. before cleaning, and the gain due to the cleaning of one 20-in. main is estimated to be from 2 to 3 lbs., the actual loss with the same flow under present conditions being from 15 to 16 lbs. For the preliminary study simultaneous measurements were taken in all three of the mains at the time of average daily flow. Previous to these tests, the 30-in. Venturi meter on the supply line and the pitometers used in the work were checked against each other, the pitometers being found to read about 1.5 per cent in excess of the meter reading. The results of the tests on the mains have been summarized and are given in Table I, together with certain comparisons that relate to age, carrying capacity and conditions of these supply mains at present and when tested.

rate. At an average annual rate of increase of 500,000 gals. per day in five years the additional amount required would he 11.63 m.g.d and 18.4 m.g.d., respectively, for average and mixmum drafts. Therefore, if the above assumptions are correct and if the present system could be restored to its original capacity, it should be good for five years more, even with a rate of deterioration twice that usually assumed.

If a new supply pipe was laid it would be at least 36 ins. in diameter and about 33,000 ft. long. At a minimum price of $8.25 per linear foot, this line would cost about $270,000, the interest on which at 5 per cent simple interest would be $13,500 per year, and at compound interest the charge would be $74,500 in five years.

The preliminary estimate for cleaning three miles of 30-in. and six miles of 20-in. pipe was $15,300, a little more than the interest for one year on the amount necessary to lay a new 36-in. main. If, therefore, the construction of the 36-in. main could be put off for five years without detriment to the service, the saving to the city was estimated to be about $60,000.

TABLE II.—DATA ON MECHANICAL CLEANING OF WATER MAINS IN SEVERAL CITIES.

City.	Sizes cleaned, ins.
Cincinnati, O.	6 to 16*
Rochester, N. Y.	6 to 20†
Newton, Mass.	24
St. Louis, Mo.	15 and 20
Brooklyn, N. Y.	16 to 36
Atlanta, Ga.	30
American Water Works & Guarantee Company	4 to 20‡

*Third contract for work.
†About 25 miles cleaned in all.
‡50 miles cleaned in 1912.

On account of the benefit which would result to the city if it were possible even partially to restore the carrying capacity of the existing mains by cleaning them, information was sought from various cities where the process had been applied. Very courteous and complete replies were received, and there seemed to be a unanimity of opinion that the work done was successful in operation and satisfactory in result. Some of the cities from which replies were received, with data on the work done, are shown in Table II.

TABLE I.—TEST AND OTHER DATA ON THREE WATER SUPPLY MAINS AT HARTFORD, CONN.

			Coef. for Kutter's formula as per Weston.			Discharge in m.g.d. Loss of head, 18 lbs.			Age	
	Diameter, ins.	When laid.	Existing.	According to age.	New laid or cleaned.	Present.	New laid or cleaned.	After 10 years.	As per coef.	Actual.
	30	1896	81.6	91.5	112	8.5	11.7	10.2	28	16
North	20	1875	63.4	75.7	110.8	2.4	4.2	3.7	65	37
South	20	1867*	74.0	75.0	110.8	2.8	4.2	3.7	40	25-40
						13.7	20.1	17.6		

*Date of laying south 20-in. main is date original main was laid.

ECONOMIC CONSIDERATIONS.

From Table I it was seen that the coefficients, third column, for the 30-in. and the north 20-in. mains were unusually low, while the south 20-in. main was in about the condition that might have been expected. If these pipes were cleaned, it was estimated that the present system should be capable of delivering 20,000,000 gals. per day to the city with the same loss of head that then existed, 18 lbs. at the City Hall; and, if not cleaned, it appeared that under present conditions of demand the construction of a new main would be an immediate necessity.

Using the consumption figures from the Venturi meter record for June, 1912, the following estimate of future consumption was made: Average rate, 9,130,000 gals. per day and maximum rate, 11,500,000 gals. per day, or 58 per cent excess maximum to average

The list prices quoted by the National Water Main Cleaning Co., of New York, for doing work in the distribution system were:

LIST PRICES PER LINEAR FOOT OF WATER PIPE FOR MECHANICAL CLEANING.

Diam of pipe.	Cost per ft., cts.
6-in.	16
8-in.	17
10-in.	18
12-in.	22
16-in.	26
20-in.	30
24-in.	40
30-in.	65
36-in.	80

These list prices were stated to be for average conditions for lengths of five miles or more only for purposes of preliminary estimate, and were submitted with the reservation that local conditions might cause considerable variation either way. In Hartford a price of 28 cts. per linear foot for 20-in. pipe was given for a 3-mile contract, with a further reduction if a greater length was cleaned. The conditions were exceedingly favorable for a large part of the way on account of few consumers on the line, advantageous location of gate valves and blow-offs for cutting out sections of proper length and also because of a parallel main with cross-connections which gave ample water for operating the machine without interference with the city supply.

A contract was entered into Sept. 4, 1912, with the National Water Main Cleaning Co. to clean, on trial, 3 miles of 20-in. pipe, and if satisfactory results were obtained the cleaning process might be continued through several miles additional of 20-in. and three miles of 30-in. pipe.

Work was begun Sept. 6 and suspended on Oct. 24 on account of scarcity of water in the reservoirs. The results were very satisfactory and during this period, 49 days, a total of 33,693 lin. ft. was cleaned. On this section there were 154 service pipes which were shut off during cleaning and only four were at all interfered with by the cleaning operations. Three of these were extension meters located at the street line with no curb cocks, and it was necessary to remove the meter and clean out the dirt. The other service affected was plugged, but was easily relieved by a force pump.

The usual force employed on this work was a superintendent, a foreman, a calker, 14 laborers and a double team for carting pipe, materials and supplies.

Under average conditions 3,000 lin. ft. was found to be the maximum effective length for cleaning. The contractor stated that 5,000 ft. had been successfully cleaned by him elsewhere, although in some places it had been possible to go only 1,500 ft. at a time, using water to drive the machine. If the machine is drawn through by a cable, the length of section is from 500 to 1,200 ft. It is stated that the machine can be operated by water under heads of as low as 10 or 12 lbs. The least available head on the Hartford lines was somewhat greater than this.

DESCRIPTION OF MACHINE USED IN CLEANING MAIN.

The pipe machine used in cleaning the mains consists of three distinct sections attached to each other by a flexible joint in the connecting shaft, which is central in all of the sections. These joints move readily and allow the machine to pass through ordinary water pipe curves if the radius is not too short. The first sections are armed with saw-tooth blades that project at right angles to the axis of the machine. The function of these blades is to cut up and tear loose the incrustation and scale that adheres to the pipe walls.

The second section is made up of two spiders armed with smooth scrapers. The function of these blades is to scrape away all matter left clinging to the pipe by the first section.

Behind these two sections is a double piston fitting the pipe very closely, with leather gaskets pressed out by steel springs which are continuous around the entire circumference. The pistons are fitted with dampers which can be adjusted to the pressure of water and so regulate the quantity needed to wash ahead the scale and other matters cleaned from the pipe. Behind each spider and loosely fitting on the center shaft are three somewhat flexible metal disks whose function it is to deflect the wash water towards the pipe walls and out through the cleaning blades and keeps the whole mass moving along with the machine.

Behind the pistons is another section which carries rows of right and left offcast scrapers. These are very heavy and are designed to smooth off the surface and leave a clean, smooth waterway. These blades are allowed only just to touch the pipe walls, their position being kept by case hardened rollers that run on the pipe wall. The gross diameter of the machine used in the 20-in. mains is $21\frac{1}{2}$ ins., the steel springs letting down at entrance and so keeping the machine firmly centered in the pipe. The steel blades are made especially for this purpose, and are nearly all replaced after each draw. This replacement, of course, depends largely on the character of the incrustation, and when mud alone is found the blades may require very little renewal.

Previous to the cleaning work, a very thorough survey was made of the capacity of the pipe line by the use of the Pitometer, and similar tests were made after the work was finished. A typical summary of the results of this work follows:

Pipe cleaned—North 20-in. main, Farmington Ave.

Section—Reservoir No. 1 to Vanderbilt Hill.

Length—First test, 16,400 ft.; second test, 14,650 ft.

Pipe laid in 1876; age, 37 years; kind, cast iron.

	(1)	(2)
Date of test	Aug. 22, 1912	Oct. 2, 1912
Make of gage	Crosby	American
Unit of gage	1-lb.	1-ft.
Head lost	20 to 55 ft.	20 to 50 ft.
Range of velocity (ft. per second)	1.69 to 2.40	2.84 to 3.97
Possible error	Per lb., 3.5%	Per ft., 1.5%
Mean coefficient in Chezy formula	69	111
Increase in capacity	61%	....
Average coefficient in Chezy formula	Weston (37y.), 75	(new) 111
	Williams & Hazen (37y.), 76	(new) 116

The loss of head was about the same in both cases, and sufficiently large so that an error of 1 lb. or 1 ft. would be comparatively negligible.

The north 20-in. main was cleaned from the reservoir to Union Place, a distance of about five miles. The south 20-in. main has been cleaned from the reservoir east about $1\frac{1}{4}$ miles. Tests of the flow made before and after cleaning indicated that in both pipes the carrying capacity had increased sufficiently to compare with Weston's tables for new 20-in. pipes. This is equivalent to a coefficient of 111 in the Chezy formula, the resulting increase in capacity being 61 per cent and 50 per cent, respectively, for the north and south 20-in. mains.

Several readings of the amounts of water used during the cleaning were made from the records of the Venturi meter at the reservoir gatehouse, and the following quantity is believed to be a reliable estimate of the amount per linear foot:

	Gals.
Quantity required to fill 1 ft. of 20-in. pipe	16.3
Quantity used in cleaning	24.0
Adding 25% for emergencies, etc.	9.7
Total estimated amount per lin. ft.	50.0

The total distance cleaned was 33,093 lin. ft. and it is estimated that 1,655,000 gals. of water were used. This amount is about 1/5 of one day's supply to the city. Table III presents a summary of the work done and the results obtained.

On account of a suggestion that the cleaning of water mains might have effect in causing or aggravating the so-called "red water" trouble, samples from the cleaned and uncleaned mains were sent to Jas. A. Newlands, Chemist of the Connecticut State Board of Health, with the request that especial attention be given in his examination to detecting any indications of "red water." The following extracts are given from Mr. Newlands' reports on the samples submitted:

Report on sample No. 1352; sample taken Oct. 1, 1912, from 20-in. main that had been cleaned:

This sample, which I understand is taken from the clean main section, shows only very slight differences from sample No. 1353, taken from the uncleaned main, and these differences are so slight as to be within the limits of laboratory error, and the slight variation which might be expected in two samples from different points on the same pipe line. On a long line of cleaned pipe we might expect that where the inner surface has been cleaned down to the iron, at such points there would be some reductions of the oxide of iron by the action of decomposing algae growths and other organic matter. This would result in iron being taken up by the water and a consequent increase in color. As the amount of dissolved oxygen in this water is probably high enough at all times to supply the oxygen required by the organic matter, there probably will be no appreciable effect on the color and iron content of the water. These samples show none, the color being the same in both cases, while the iron amounted to 0.3 part per million in this sample and 0.2 part in sample 1353.

Letter of Oct. 7, 1912: With reference to the tap and hydrant samples, I will say that the results obtained do not show any appreciable effect of the cleaned main (on) the appearance or quality of the water. Under some conditions, as I have suggested, it is quite possible that for a time there might be some increase in color and iron matter content, due to the action of decomposing organic matter on the fresh iron with which it comes in contact; but the amount of organic matter in your reservoirs does not usually become great enough to exhaust the oxygen dissolved in the water, and I do not anticipate any material effect on the color of the water. However, it is difficult to determine how the various types of water will act under varying conditions, and I think it will be worth while to keep track of the relative colors of the waters in the cleaned and the uncleaned mains as the work goes on.

Report on sample No. 1378, taken Oct. 28, 1912, from south 20-in. main that had been cleaned.

These results show practically no differences in the physical appearance of the chemical constituents of the water as compared with sample No. 1379, taken from the north 20-in. main. No differences were noted in the color, turbidity or sediment of the two samples, although we might expect slight differences even in two samples taken from different points in the same main. There is no indication at this time that cleaning the main has resulted in the beginning of "red-water" trouble.

Letter of Nov. 4, 1912. I have looked the

TABLE III. SUMMARY OF WORK DONE AND RESULTS OBTAINED IN MECHANICAL CLEANING OF WATER MAINS AT HARTFORD, CONN., IN 1912.

Northerly 20-in. Main.

Location—	Date tested.	Length, ft.	Range of flows, m.g.p.d.	Lost head, per 1,000 ft.	Chezy coefficient.
Reservoir to Le May's:					
Before cleaning	8-31-12	5,150	3.3 to 4.7	2.90 to 7.65	60 to 67
After cleaning	9-21-12	5,450	3.6 to 5.2	0.64 to 2.09	124 to 159
After cleaning	9-26-12	5,450	2.5 to 5.3	0.96 to 2.57	109 to 125
Le May's to West Hartford Center:					
Before cleaning	9-14-12	4,430	2.8 to 4.2	3.80 to 7.00	50 to 56
After cleaning	9-26-12	3,750	3.6 to 5.2	1.70 to 2.90	94 to 115
West Hartford Center to Highland St.:					
Before cleaning	9-10-12	6,420	3.1 to 5.8	2.03 to 8.46	69 to 75
Vanderbilt Hill to Broad St.:					
Before cleaning	10- 3-12	10,180	2.4 to 4.3	1.81 to 5.81	61 to 65
W. Hartford Center to Vanderbilt Hill:					
After cleaning	10- 2-12	5,450	4.0 to 5.6	1.18 to 3.63	110 to 112
Reservoir to Highland St.:					
Before cleaning	9-27-12	16,300	2.4 to 3.4	1.40 to 2.89	67 to 70
Reservoir to Vanderbilt Hill:					
After cleaning	10 -2-12	11,650	4.0 to 5.6	1.55 to 3.01	110 to 112
Southerly 20-in. Main.					
Reservoir to Highland St.:					
Before cleaning	8-27-12	16,300	2.58 to 2.91	1.52 to 3.55	73 to 77
Before cleaning	10-11-12	16,300	3.25 to 4.08	2.17 to 3.41	76.3 to 76.[illegible]
Reservoir to Westland St.:					
After cleaning	10-28-12	6,400	4.1 to 5.4	1.61 to 2.70	110 to 119

samples over carefully for evidence of "red water" trouble, but there appears to be no such effect at this time. The results on the two samples are more nearly alike than we usually find them, as it frequently happens that samples, although taken from two points on the same main, will show some differences in the amount of turbidity and sediment, especially, due to unavoidable differences in the amount of sediment stirred up when the samples are taken. There is no evidence in these results, therefore, to show that the cleaned main has any effect on the appearance of the water at this time.

Some criticism of the work was made by a writer in one of the local newspapers that the cleaning of the pipes destroyed the inner coating, and the intimation was that the rusting of the mains would probably proceed very much more rapidly in the future than in the past. It has been stated by some engineers that the cleaning of cast iron pipe frequently does increase the rate of tuberculation and under some conditions that may be true. Previous to undertaking the Hartford work, inquiry was made concerning this matter of various water departments where the pipe had been cleaned. Very little information was obtained regarding the rate of returning incrustation, but regarding the removal of the coating the opinion of all who had made examination was that the inside of the pipe was uninjured. Observations made on the ground showed no evidence of damage of this kind. The hardest test probably was encountered when the machine was pulled into the 9-ft. piece of pipe made ready for insertion in the line. In several cases this section was examined and no removal of the coating was apparent.

The incrustation removed was in places over an inch thick nearly around the pipe. Besides this, considerable quantities of pipe moss were found near the reservoir end of the mains. Velocity measurements across a section of the pipe were taken before and after cleaning. Defining the cofficient of sectional velocity as the quotient of the mean velocity divided by the center velocity, this figure was 0.66 before cleaning and 0.84 after cleaning.

Design of Proposed Aeration Basin for Water Supply of Arma, Kansas.

(Staff Article.)

The aeration basin here described is an interesting feature of the proposed new water supply system for the city of Arma, Kans. The supply will be drawn from a deep well and pumped through a 6-in. discharge line to the aeration basin. A suction main will connect the aeration basin with the high service pump.

As shown in Fig. 1 the aeration basin is 40 ft. in inside diameter and 9 ft. deep. The wall is 12 ins. thick and the floor is 9 ins. thick. The basin is of reinforced concrete construction throughout. The detail of the wall section and reinforcement is shown in Fig. 2. The bottom of the basin is reinforced with ½-in. round rods spaced about 18 ins. on centers in both directions. The wall is 18 ins. wide at the top and is given a 6-in. coping 12 ins. deep. Below this point the wall is 12 ins. thick to the floor line below which a footing 30 ins. wide is to be constructed. The coping is reinforced with ½-in. round rods as shown in Fig. 2. A pipe railing, consisting of 1½-in. pipe posts set 12 ins. into the concrete and spaced 8 ft. apart, and two horizontal lines of piping of the same size will be placed upon the top of the basin wall. The two rails will be 18 ins. apart and the lower one will be 18 ins. above the wall top.

For the inside and outside wall forms shiplap or other tight-fitting lumber will be used. Concrete will be 1:2:4 mix, using either crushed limestone or Joplin chats for the aggregate. If the chats are used tests will be made to determine the proper proportion of cement to use. All reinforcing rods will be given a lap of at least 20 ins. and the ends of all rods will be bent to an angle of 90° about 6 ins. from the end.

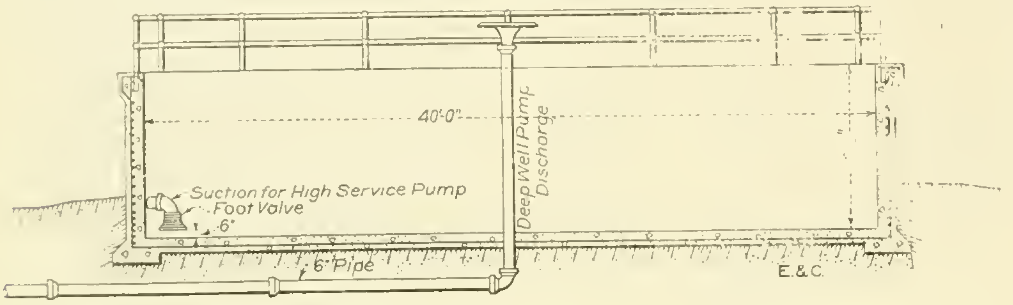


Fig. 1. Sectional Elevation of Proposed Aeration Basin for Water Supply of Arma, Kansas.

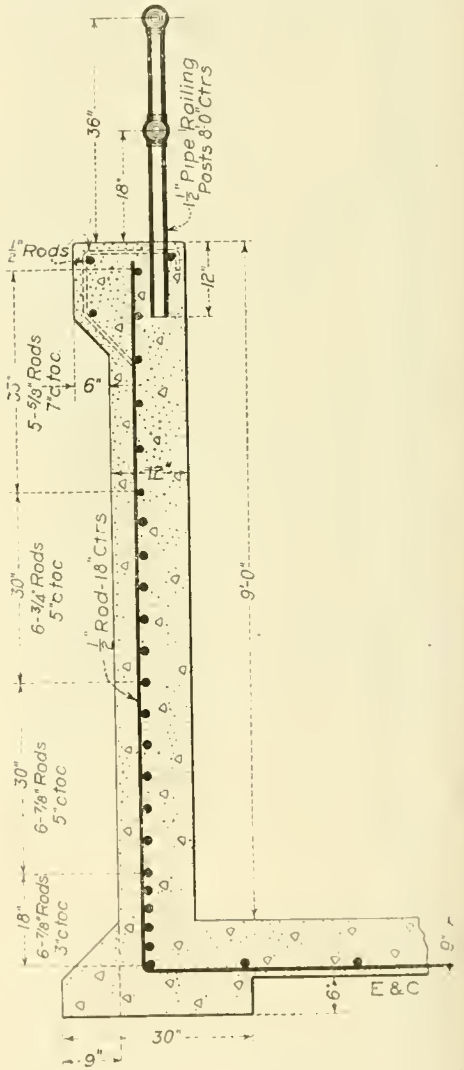


Fig. 2. Detail of Aeration Basin Wall, Arma, Kansas.

The design here illustrated was prepared by Mr. Albert C. Moore, Consulting Engineer, Joplin, Mo. The design was approved by Mr. C. A. Haskins, Sanitary Engineer, of the Kansas State Board of Health.

Relative Cost of Hand, Steam and Air Drill Work in Boston City Trenches.

The unit costs of drilling ledge and boulders in trench work for sewers, water pipes, etc., and other construction work in city streets have always been high, because it is seldom economical to install a steam power plant and piping for such small jobs, and drilling by hand in a large city where wages for labor are high is necessarily expensive. The advent of a portable air compressor, operated by electricity or gasoline engine, not too heavy to be moved easily from street to street, has seemed to solve the problem of accomplishing this class of work economically.

Based on hand-drilling costs, a standard rate of $4 per cubic yard has been allowed for rock removed within trench limits on water-main installation work in Boston, Mass. The amount allowed in solid rock per running foot in ordinary trenches for water pie is 15 cu. ft. Actual costs of this Boston city work, as given to the Compressed Air Magazine by Frederick I. Winslow, an engineer in the Sewer and Water Division of the Public Works Department, are as follows:

With a hand-drill the number of feet drilled in a working day of 8 hours varies from 8 to 12 ft., the cost per day of the drill gang is $7, the cost per lineal foot drilled about 87 cts., and the cost per cubic yard of rock excavation about $3.50. A steam-operated drill is capable of 50 to 70 ft. per 8-hr. day, the cost of operation $12 per day, the cost per lineal foot about 24 cts., and the cost per cubic yard of rock excavation about 96 cts. A compressed-air drill is able to drill from 80 to 120 ft. per day, and the cost of operation, including an allowance for depreciation, repairs, moving, etc., is $16 per day, the cost per lineal foot about 20 cts. and the cost per cubic yard of rock excavation about 80 cts. The difference in favor of the compressed-air drill is greater than the figures show, according to Mr. Winslow, on account of the greater ease of manipulation, and the fact that the compressor can be operated by unskilled labor.

The above costs for steam and compressed-air drills are based on but one drill operated from the power plant, although, of course, either type of plant is capable of operating other drills; with the small portable air compressor used two drills may easily be operated simultaneously.

Water Supply on the Rand.—A proposed new water supply for South Africa is described in the Daily Consular and Trade Reporter as follows:

"With the constant growth of population of the city of Johannesburg and the surrounding towns of the Witwatersrand, and with the increased demand for water for use of the gold-mining industry, there has been a great dearth of water for several years, the present supply being entirely inadequate, especially during those seasons when the rains have not been up to the average. The Rand water board, which has charge of the water supply for practically all of the Rand district, has now approved a scheme for securing an adequate additional supply of water from the Vaal river. This project will take four years to complete and will cost approximately $6,500,000. The proposal is that the water be pumped from the river in the vicinity of Vereeniging to the existing water board pumping station at Zwartkopjes, and from that point it will be passed through pipes to balancing tanks and reservoirs already in use in the distributing centers. The Vaal River has for a long time been looked upon as the natural source of supply to meet the Rand water problems. Much delay, however, in bringing the present scheme to a head has been caused by lengthy negotiations with those farmers and others who hold riparian rights on its banks.

"While the details are not yet definitely settled, it is probable that works will be established along the river bank in the neighborhood of Vereeniging, where a power station will be installed. It is estimated that this new source will supply 10,000,000 to 12,000,000 gals. daily. Steps are to be taken at once to secure the necessary legislation to expedite the work."

Buildings

Construction of the Caisson Foundation for the J. P. Morgan & Co. Building, New York.

(Staff Article.)

The foundation for the new J. P. Morgan & Co. office building, located at the corner of Broad and Wall Sts., New York, presents some interesting and unusual features. The work consisted of sinking to rock 18 pneumatic caissons, joined together at their ends to form a continuous cofferdam wall 7 ft. thick, and 19 interior piers sunk in the open after the outer wall was completed. Figure 1 shows a plan of the building site, with general dimensions.

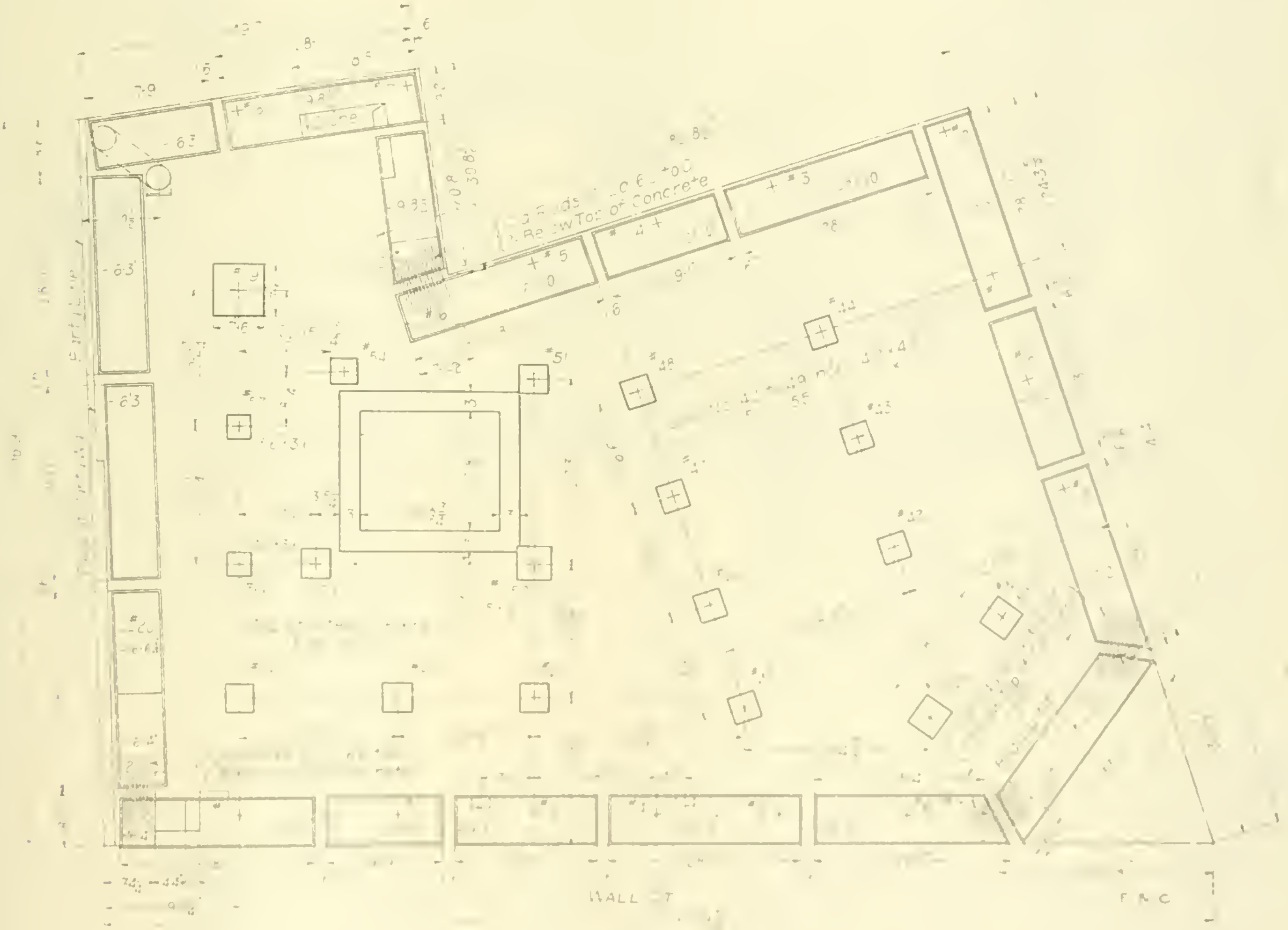


Fig. 1. Plan of Site of the J. P. Morgan & Co. Building, Showing Dimensions of Lot and Position and Size of Each Caisson.

The drawing also shows the size and location of each caisson.

The foundations have been designed to carry a much greater load than that of the present four-story building, in fact an increase in height of 20 stories may be made without strengthening the substructure. The excavation for the sub-basement is very deep, being 52 ft. 9 ins. below the curb. The basement level is 16 ft. below the curb, while the [illegible] cofferdam wall around the lot [illegible] bonded to the rock to make a [illegible] basement.

PRELIMINARY WORK AND UNDERPINNING ADJACENT BUILDINGS.

Before sinking the caissons [illegible] work on the foundation for the new building it was necessary to shore [illegible] walls of the adjacent Mills Building, which bounded the lot on two sides. These walls were very heavy, weighing about 16 tons per linear foot of wall. They rested partly on piles and partly on the soil, which was 16 to 18 ft. below the curb. These conditions presented a very difficult problem, the underpinning operations requiring special care. The boilers, machinery and piping in the basement of the Mills Building prevented any work being carried on inside that building, and all operations had to be made from the basement of the old buildings on the Morgan site. These buildings, which included the Drexel Building and the Mechanics & Metals Bank Building, were torn down at the same time that the shoring and underpinning operations were carried on; thus no time was lost in waiting for a clear site.

The underpinning was accomplished by sinking 19 pneumatic cylinders, spaced about 12 ft. apart, to bed rock, which lies at a maximum depth of about 65 ft. below the curb. These cylinders are 48 ins. in diameter, and are made by bolting together 1 ft. and 6 ft. cast iron sections having flanged ends. The cylinders were jacked down, and the excavation within was made under air pressure, the maximum pressure being [illegible] lbs. per square inch. The two bottom sections of the cylinders are 6 ft. in length, one [illegible] the air lock and the other, which has a cutting edge, for a working chamber. The cylinders were [illegible] [illegible] bed rock [illegible] concrete was then brought up to the underside of the [illegible] and rails and grillage beams were placed under the cylinders. Steel wedges were used to force the [illegible] tightly against the [illegible] beams. The cylinders were then filled with concrete up to the underside of the original wall, large grillage beams 7 ft. long being placed across the top of the cast-iron cylinders to give the wall an added bearing. Much difficulty was experienced in sinking some of these cylinders, due to the wood piles which formed part of the original foundation for the walls of the Mills Building. As the excavation progressed these piles were sawed off in short sections and were hoisted through the air lock.

EXCAVATION AND CAISSON WORK.

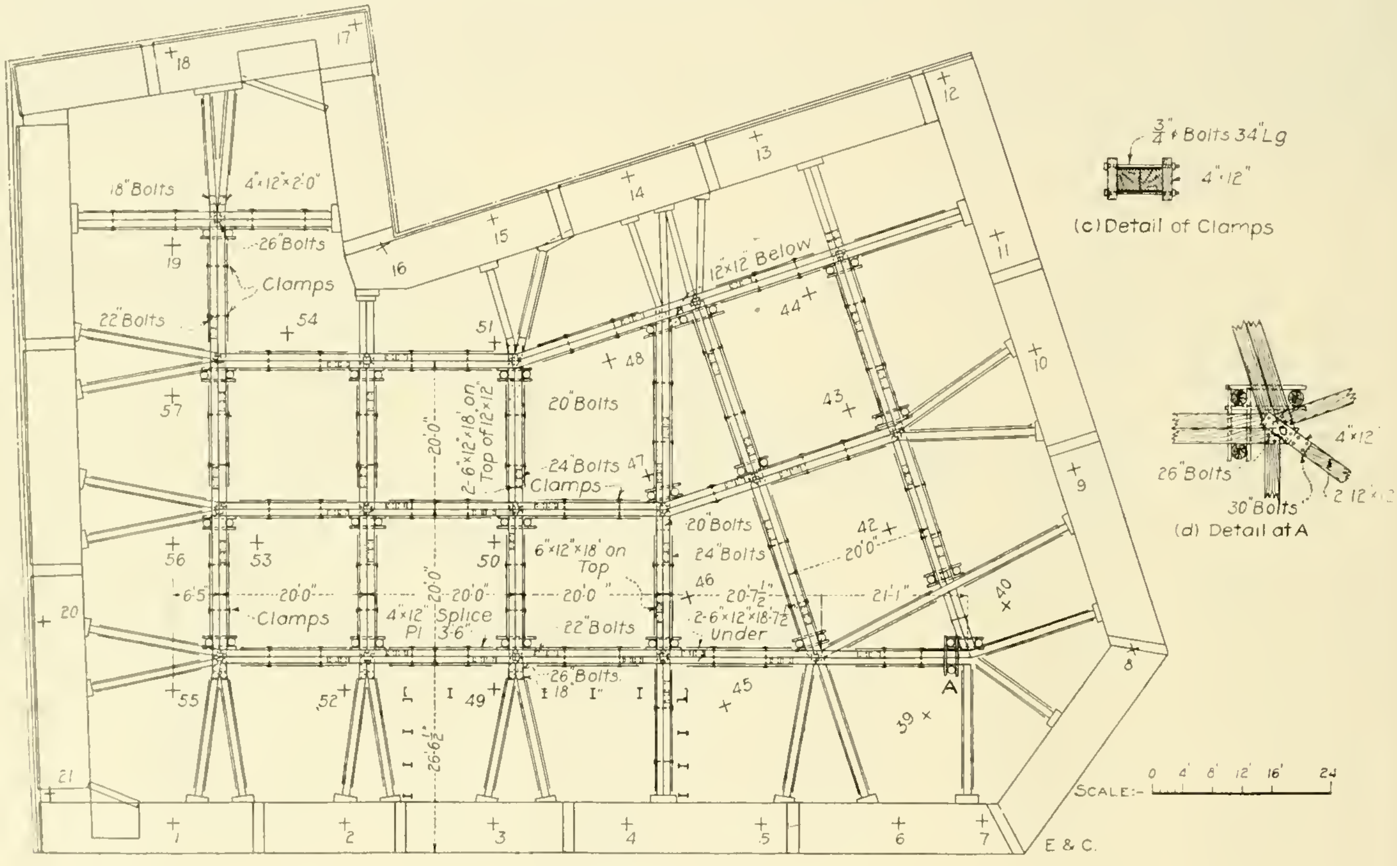
After the underpinning had been finished the site of the new building was excavated to a depth of 13 ft. below the curb, and the caissons, starting from that level, were sunk through a layer of quicksand and hardpan to rock. The average depth of the rock was 60 ft. 2 ins., the greatest depth being 65 ft. 5 ins. below the curb. The maximum air pressure used was 30½ lbs. per square inch. The layer of hardpan varied in thickness from 5 ft. to 19 ft. 2 ins. Clam shell and Steubener buckets, operated from four stiff-leg derricks, were used for the excavation work. The interior piers were then sunk to bed-rock by the open method, starting from a plane 52 ft. 9 ins. below the curb.

The sides of the caissons were formed of [illegible] sheeting. The working chambers were built of concrete, reinforced with steel bars, concrete being added in sections as the sinking progressed. The weight of the caissons was not sufficient to overcome the uplift of the air pressure and the skin friction on the sides of the caissons, and it was necessary to add from

300 to 600 tons of cast-iron blocks to each caisson to sink it. The caissons were all 7 ft. 4 ins. wide and varied in length from 16 ft. 9 ins. to 28 ft. 11 ins. They were sunk 1 ft. 6 ins. apart and later were joined together by filling the spaces with concrete, deposited by the pneumatic process, thus forming a continuous wall around the entire lot (see Fig. 1). Within this wall the lot was excavated to a depth of 52 ft. 9 ins. below the curb.

BRACING FOR COFFERDAM WALLS.

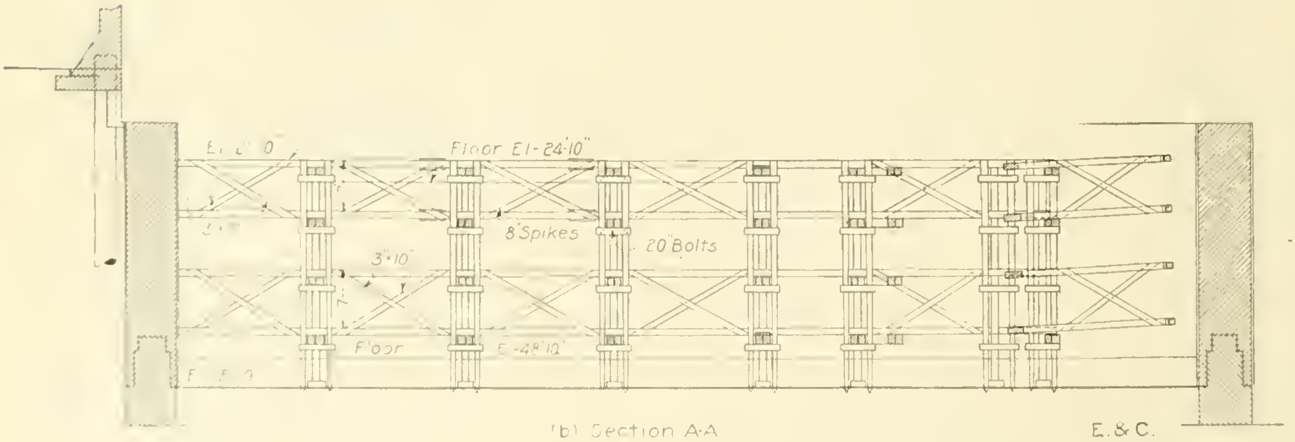
As the excavation progressed heavy lateral bracing consisting of 12x12-in. timbers, was placed across the lot to withstand the pressure from the adjacent soil and buildings. Figure 2 (a) shows a plan of this bracing. This drawing shows the arrangement and position of the timbers and indicates the position and type of splices used. Figure 2 (b) shows a longitudinal section of the bracing. It also shows the pile supports at the connections and indicates the type of vertical bracing used. Figure 2 (c) shows a detail of the clamp used to fasten timbers together, and Fig. 2 (d) indicates the method used to connect the members at the panel points.



(a) Plan of Bracing

(c) Detail of Clamps

(d) Detail at A



(b) Section A-A

Fig. 2. Plan, Section and Details of Bracing for Cofferdam Walls of the J. P. Morgan & Co. Building.

ARRANGEMENT AND TYPE OF EQUIPMENT USED.

In placing the concrete in the caissons the Lakewood system, consisting of a steel tower and chutes, was used. The tower was erected near the center of the lot, the concrete being mixed at the base of the tower and hoisted to the charging hopper, where it was dumped into chutes leading to the caissons. A four-boom steel traveler was erected at the west end of the lot, but only three booms were used for handling the buckets to and from the air locks. Where convenient, the concrete chutes were fastened to the sides of the steel traveler. In addition to this equipment, two stiff-leg derricks were used at the east end of the lot.

Two wooden hoppers were erected, one on Wall St. and the other on Broad St. Into these hoppers was dumped the spoil excavated from the caissons, and from there it was dumped into the trucks placed in the streets below. A timber platform was constructed along the center line of the lot, leading from Wall St. Two additional wooden hoppers were built on this platform to hold the excavated material. Trap-doors were cut through the platform, and all sand, stone and cement received was dumped into the bins, from which they were fed into the concrete mixer. Both the concrete mixer and the engine for operating the lifting device of the tower were placed under the platform, and the whole arrangement was such as to give the quickest and most economical concrete mixing layout. The booms of the steel traveler and the two stiff-leg derricks could reach every point of the lot; therefore it was not necessary to shift the derricks at any time during the sinking of the caissons. A hospital air lock was fitted up on Wall St. to take care of any possible cases of the "bends," but it was not used at any time during construction.

The entire foundation work for the J. P. Morgan & Co. Building, including the underpinning of the adjacent buildings, was done by The Foundation Co., New York, under the personal direction of Mr. Louis L. Brown, vice-president. The waterproofing of the walls and floors is being done by The Waterproofing Co., New York. Trowbridge & Livingston are the architects, and Mr. Daniel E. Moran is the consulting engineer for the owners. Marc Eidlitz & Sons are the general contractors. We are indebted to the Foundation Co. for the data contained in this article.

Reinforced Concrete Columns; Some Considerations Affecting Their Design.

In the hope that the discussion would bring out some essential data relative to the design of reinforced concrete columns a paper, consisting of a series of ten questions with brief discussion, was presented before the Western Society of Engineers, on Feb. 9, 1914, by Mr. Ernest McCullough. The author gave a list of books which he thought contained the most essential data on the subject. We have purposely omitted mention of these books, as it seems to us that what is needed are new data, not merely a discussion of those already published. The great differences in unit stresses allowed by the building codes of 20 cities and by the Joint Committee Report was shown by the written discussion. The series of questions presented, together with some comments by the author and some of the discussion, is herewith given:

(1) What is the cross sectional area of a reinforced concrete column? Commercial designers, almost to a man, use the cross sectional area of the column complete. Others omit a skin ½ in. thick over the entire surface, but place the steel at least three times that distance from the surface, for fire protection. Others count none of the concrete as effective that is not enclosed within the upright reinforcement. Some building ordinances are so worded that any one of these assumptions may be used.

(2) What is the maximum compressive stress advisable on a column? Experiments have shown that no evidence of flexure appears in columns having a diameter less than one-twentieth the length, but neglect to say whether the diameter is gross or net. Few ordinances permit a reinforced concrete column to be erected with a length exceeding twelve times the diameter, again omitting to state whether the gross or net diameter is meant. Granting that no bending occurs what should be the maximum stress? Commercial designers almost to a man use 750 lbs. per square inch on the gross section.

(3) Should the verticallly reinforced column be used? American experiments seemed to indicate in one or two instances that horizontal ties around vertical rods were of no greater value to prevent buckling than the tensional strength of the concrete. American practice in the design of such columns proceeds upon the theory that such ties are for the sole purpose of assisting in preventing buckling of the reinforcement. If the Joint Committee Report is correctly read such ties are not required for any other purpose than to hold the steel in place during construction.

European experience and experiments indicate that while such ties have a value to prevent buckling of the steel, a distinct gain is made in strength by their use which is contrary to the general belief in America. The French Commission found that the spacing of these ties and the size of the wire affected the ultimate resistance of the concrete so materially that a formula was prepared into which a letter representing the volume of steel in ties was introduced. The Royal Institution of British Architects adopted rules based on those of the French Commission. A development of the formula gradually leads to the Considere formula for hooped columns, wherein the strength of the column is based on the fact that steel placed spirally as a hooping is 2.4 times as effective as the same volume of steel placed vertically.

The concrete column with [illegible] reinforcement, with ties only, [illegible] suspicion and a free discussion [illegible] invited. If it should [illegible] the sooner we know this the [illegible] are thousands in use over the [illegible] designed with a stress of 750 lbs. per [illegible] over the gross horizontal section [illegible] nominal ties intended to serve no [illegible] than to act as stiffeners to [illegible] the vertical steel from buckling.

4. Hooped columns should [illegible] the amount of steel limited and where should the limit be set? To Considere is given the credit for developing this column and he advised the use of mild steel for hooping. Professor Withey of the University of Wisconsin found that the elastic limit of the steel hooping was reached about the time the concrete was stressed to its ultimate strength so he advised the use of steel having a high yield point. This seems logical since the only reason for hooping is that it adds to the ultimate strength of the concrete.

5. Are hooped columns dangerous from the standpoint of fire risk, when their strength depends upon the hooping?

6. Are the present methods of fixing the pitch of the hooping adequate? The writer proposes a pitch of one-sixth the core diameter for columns having cores less than 18 ins. in diameter; one-eighth the core diameter for columns having cores from 18 to 24 ins. in diameter; one-tenth the core diameter for columns having cores more than 24 ins. in diameter; provided, that the pitch shall in no case exceed 3 ins.

7. The writer has seldom seen a design of a reinforced concrete building in which the effect of wind on the structure seemed to have been considered. Is this typical of reinforced concrete buildings and why?

8. Some building ordinances state that unbalanced moments carried to columns must be provided for. How often is this done? The writer has not known of its being done, but there may be a few high minded designers who do look out for this point. The writer proposes the following clause for all city ordinances: "For unbalanced moments carried into columns there shall be added to the load, for which the column is to be designed due to dead and live loads, the following percentages: To the columns supporting the roof, 30 per cent, to the next lower tier, 25 per cent; with a progressive reduction of 5 per cent on each floor to 10 per cent of the total load, which 10 per cent shall be added on each floor thereafter to the first. This load shall not be counted in designing footings and shall be in addition to wind load. In addition to this increment wall columns to be designed for full live loads."

On the subject of unbalanced moments American text books are singularly silent, but a good discussion is to be found in the book by [illegible] & Bowie.

9. How should columns be carried up through buildings? It is convenient to have the steel rods one story in length and to place stubs at the floor levels for the purpose of delivering stress to the next section. Of what value are these stubs and how far should they extend above and below the floor level? Some engineers consider these stubs as nothing more than vertical pins, assisting to a slight degree perhaps in case of heavy winds. Other engineers proportion them on the assumption that the stress travels through the concrete to them from the steel on the floor above to that in the floor below. Has this assumption any basis in fact?

10. Is it right to place the vertical steel in the concrete without providing plates at the lower end and caps over the upper end to distribute the load over a larger area of concrete than is provided by the end of the steel? The steel is assumed to be carrying throughout its length fifteen times the stress carried by an adjacent equal area of concrete, the bond stress throughout the length being constant. When the steel suddenly ends, with the concrete carrying its full load, what is the effect on the surrounding concrete when no attempt is made to transmit the stress directly through from end to end of steel, but on the contrary one bar may be some inches away from the parallel bar in the floor above, or below? Should there be plates at each floor and should there be plates in the footings? The writer recently examined plans prepared in the offices of engineers in high standing in which the lower ends of the column rods were hooked under the mat of steel in the footing. What is the general opinion of such practice?

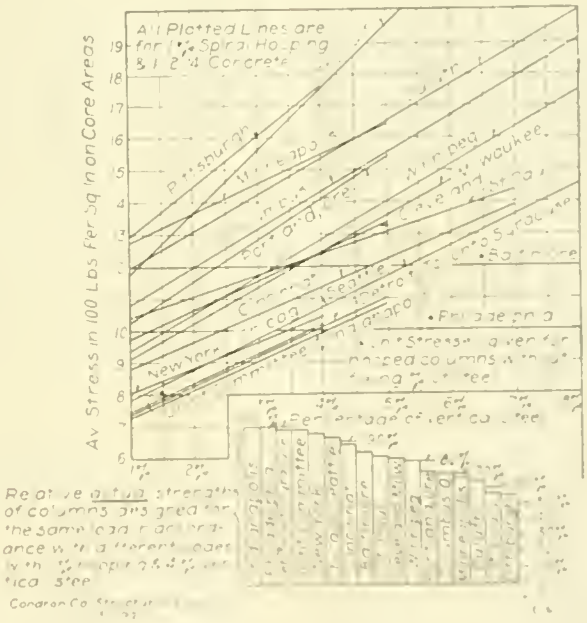


Fig. 1. Unit Stresses in Reinforced Concrete Columns Allowed by Building Codes of 20 Cities and by Joint Committee for 1:2:4 Concrete, 1 Per Cent Spiral Hooping and Different Percentages of Vertical Reinforcement—Also Relative Strengths of Such Columns for 4 Per Cent Vertical Steel.

DISCUSSION BY MR. CONDRON

The following discussion by Mr. T. L. Condron, of Condron & Co., Chicago, Ill., presents very clearly the radical difference in unit stresses allowed by the building codes of various cities and by the Joint Committee Report and also discusses other points.

I offer the accompanying diagram (Fig. 1) as a contribution to the discussion of the paper by Mr. McCullough on "Reinforced Concrete Columns." This diagram shows graphically the comparison of allowable unit stresses and the relative strength of spirally hooped columns based upon the requirements of the building codes of 20 American cities and of the "Joint Committee on Concrete and Reinforced Concrete" of the several engineering societies. This diagram shows at a glance the inconsistency of these various building code requirements. We may suppose that the recommendations of the Joint Committee are based upon scientific knowledge of the actual strength of hooped columns, because the membership of that committee includes men who have given much study to the actual strength of concrete and reinforced concrete as determined by laboratory tests.

It will be observed that the building codes of New York, Chicago, Seattle, Detroit, Toronto, Syracuse and Indianapolis do not differ materially from the recommendations of the Joint Committee, except that the Joint Committee limits the percentage of vertical reinforcement to 4 per cent, while the Chicago ordinance permits as high as 8 per cent and the other cities, with the exception of New York, permit as high as 5 per cent of vertical reinforcement. It has been the practice in our office to design columns in accordance with the requirements of the Chicago code, making the maximum limit of 5 per cent of vertical reinforcement.

It will be noted that a column designed with 1 per cent hooping and 4 per cent vertical steel under the requirements of the St. Louis or Pittsburgh codes, for a given load, would be only rated for 56 per cent of that load in Indianapolis or Philadelphia; while on the other hand a column designed for a given load under the Indianapolis or Philadelphia codes, for a given load, would be accepted in St. Louis or Pittsburgh for nearly double that load.

No attempt has been made to show on the diagram of Fig. 1 the relative allowable unit stresses under these various codes, for other conditions than for 1:2:4 concrete with 1 per cent hooping and with varying percentages of vertical steel, but equally interesting comparisons can be made of the requirements for various grades of concrete and different percentages of hooping.

The following table shows the allowable loads, in tons, on the same column designed under different building codes. The table is for a 1:2:4 concrete column with a 20-in. diameter core, with 1 per cent hooping and with 4 per cent of vertical steel:

Building code of—	Load, tons.
Indianapolis, Philadelphia	157
Detroit, Toronto, Syracuse	159
"Joint Committee"	160
New York	164
Chicago, Seattle	169
Baltimore	189
Cleveland, Milwaukee	196
Portland, Oregon	222
Columbus, Ohio	226
Minneapolis	236
St. Louis	276
Pittsburgh	276

In choosing a formula for the allowable stress in a reinforced concrete column we can not take the average of all existing formulas as the correct one, for the data which we have would indicate that the lower and almost the lowest of these ordinance requirements are correct and that the higher permissible unit stresses are not justified by our present knowledge of the strength of hooped columns. When it is borne in mind that some commercial designers use column formulas which are quite as radical as those giving the highest value in Fig. 1 it will be seen that there is good reason to call a halt on the reckless designing of reinforced concrete columns.

I quite agree with the author that there is no justification for the use of concrete columns reinforced simply with four vertical rods and a few separated hoops or bands. Certainly such columns are dangerous to use, and it is surprising that so many of them have been used with so few disastrous results. The fact is that columns in buildings seldom receive the full load for which they are presumably designed, but there is no part of a building that should be more conservatively designed than the supporting columns.

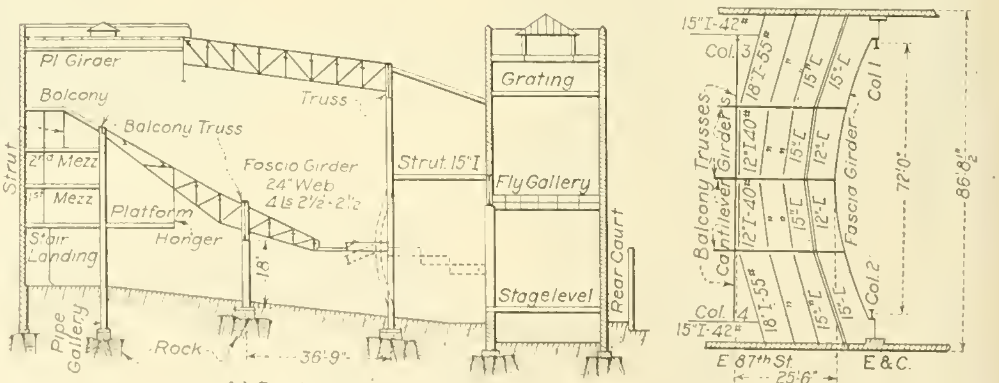
A series of reinforced concrete column tests is to be made this month at the Bureau of Standards in Pittsburgh, under the direction of the American Concrete Institute, from which we shall gain very valuable information. Until these results are announced it seems useless to discuss the theoretical strength of such columns or to recommend anything different than the more conservative formulas indicated on the accompanying diagram.

GENERAL DISCUSSION.

In the general discussion of the questions contained in Mr. McCullough's paper the lack of definite data on the subject was evident. As one engineer put it the disinclination to venture opinions proved the wisdom of the engineers present. However, some interesting, although somewhat contradictory, comments were made concerning the thickness of concrete outside of the spiral reinforcement necessary to fireproof the steel; the comparative costs in Chicago of reinforced concrete columns and steel columns incased in concrete; and the different methods in use for splicing the steel reinforcement at or near the floor lines. To meet the latter requirement the discussion brought out the fact that stubs, caps and short sections of gas pipes were being used, but the best method no doubt consists of lapping the bars a sufficient amount to develop the strength in band. The principal objection to the lapping of bars, where the upper column section is less than that of the lower, is the possibility of leaving voids between the close network of bars. A written discussion by Mr. Edward Godfrey condemning the use of vertical reinforcement without efficient spiral or hoop reinforcement was presented. (The essential points in the latter discussion are contained in an article by Mr. Godfrey, in Engineering and Contracting of Dec. 17, 1913.—Editors.)

Comments on the Cause of Failure of the Orpheum Theater, New York.

Sirs: About a year ago the Orpheum theater in New York collapsed. Only a short time ago reports were made public purporting to show the cause of that collapse. Many experts were heard from and all of the reports seem to be in, yet the engineering profession is exactly where it was before the reports were made public. It therefore seems proper to call attention to an egregious error in the design of this structure which ought to have caused a wreck exactly as this one occurred. This error was not mentioned by any of the investigators.



(a) Section (b) Plan

Fig. 1. Section and Part Plan of Orpheum Theater, New York, Showing Type of Construction and Manner of Failure.

Figure 1 indicates the construction to which attention is called, the dotted lines indicating what might be expected to happen with such construction. A curved girder with a span of 72 ft. and an offset of 12 ft. could scarcely be expected to do anything but "flop down" and bend and twist the slender column, to one corner of which it was connected. The next bend in the column would of course be at the brace, and the column would double up. This is precisely what it did. Then all of this roof load falling on the balcony would break it up considerably—which it did.

But it may be true that the little beams at the foot of the balcony cantilevers are extensions of those cantilevers and are intended to carry the fascia girder. It does not look as though they would perform this function. In any event this construction causes a very considerable load at the end of the fascia girder to be taken into the column through the medium of a wide cantilever bracket attached to the corner of a slender column which is already greatly over-stressed. When the cantilever brackets of the balcony deflect they would carry with them the fascia girder. As the ends of this girder are riveted to these cantilever brackets on the slender columns, the load on the brackets would be augmented by an unknown amount.

It would not take very much load, hanging on a 2-ft. bracket, connected to the corner of a slender column, to produce excessive stress and to bend the column. The fascia girder was not bent around the column with its web attached to it, but it was connected to the end of a separate bracket. This fascia girder simply acted as a great bender and twister on the column. When these columns failed, the roof fell, and this great weight falling on the balcony was too much for it to sustain.

This failure shows the great importance of a small portion of a design or even of a detail.

Very truly yours,
Edward Godfrey.
Monongahela Bank Building, Pittsburgh, Pa., Jan. 10, 1914.

Comments on "The Problems of the Contractor."

Sirs: The article by Mr. Wason in your issue of Jan. 14, 1914, was read with great interest and appreciation by most of your readers, I am sure.

However, what he has to say about the inspector and the superintendent moves me to volunteer a few remarks of my own.

Having been an inspector at one time I feel it my duty to speak in behalf of that unfortunate individual and to defend him, if I can, from unmerited abuse.

In the first place the inspector is sent on the job with instructions to see that the work is done according to the specifications, and he seldom is given the power to use his judgment or discretion outside of the letter of the specifications; therefore he usually feels that this important document is in the same class with the ten commandments, and that it must be enforced to the letter.

Mr. Wason is quite right in saying that the inspector must not go beyond bounds; he must not go beyond bounds in either direction —he must not dictate to the contractor's men, nor must he allow himself to be hoodwinked by them, and that is what makes his position so trying. He must have all the good qualities mentioned by Mr. Wason, and in addition must be alert and alive to the multitudinous tricks and schemes practiced by the contractor's men in skimping work and getting it passed; for it must be admitted that most superintendents are not only clever at that sort of thing, but they also thoroughly enjoy "putting it over."

Although the superintendent generally is a far better executive and is better trained in methods of construction, his technical education is often far inferior to that of the inspector, and that is where trouble arises. The superintendent treats with disdain and contempt the small but important details which it is the inspector's duty to guard; he considers them "tommyrot" and the inspector a fool for insisting upon them. They interfere with his progress report, perhaps, and that is what the superintendent cannot tolerate. I might add that the inspector is generally instructed by his superior to make suggestions, but never to give orders about the way the work should be done.

While the specifications are the guiding star for the inspector, the contractor's representative sees the sun, moon and stars revolving about the progress chart. His employer's profits are based on it, and therefore his own advancement and reward depend upon it. He is intent upon getting the work done quickly and cheaply, and if it falls down after his responsibility ceases, that is none of his affair. (I am speaking of the contractor's superintendent, not of the contractor himself.) He may have, and often does have, all the good qualities which Mr. Wason mentions, but he always has this zeal for getting the work done quickly and cheaply, and it is the inspector's office to act as a sort of governor to this zeal, to prevent it from running away and injuring the rest of the machinery. I have known two superintendents working in opposite tunnel headings deliberately to imperil the lives of their crews by trying to blast the rock in one another's faces for the mere empty honor of firing the blast that opened the tunnel. I have had to threaten a superintendent with the police to induce him to take time enough to shore up a hanging rock which was threatening momentarily to fall down on his concreting crew. I have known the superintendent of steel erection on a large building in New York City to order his men to erect the columns on concrete footings which were so rotten that they could be kicked to pieces, simply because he had his men and steel all ready to begin erection, and he was not responsible for the footings. The superintendent is generally willing to take a chance, because he may gain considerably by it; but we cannot blame the inspector for refusing to take any chances, as he has nothing to gain and everything to lose thereby.

Mr. Wason's superintendents probably are of a different order from those mentioned above, but I will venture to say that each one of them is personally far more interested in seeing that his progress report breaks all records, than that the finish and workmanship on the job shall be above criticism.

The contractor may be absolutely honest and conscientious, but the inspector needs to be everlastingly alert to prevent foremen and superintendents from subordinating the interests and rights of the owner, to their own personal ambitions to make records for economy and speed of construction.

Very truly yours,
Frank C. Boes
West New Brighton, N. J., Jan. 28, 1914.

Railways

Modern Methods of Economical Railroad Location.

Contributed by H. H. Edgerton, Jr., Assistant Engineer, Chicago Great Western R. R.

That most of the railroads of America are poorly located to serve the present traffic requirements is a fact only too patent to operating officials who know that if the general routes had been differently selected on lines capable of improvement as the increase of tonnage demanded, the roads would be successful revenue producers, even when operated under rates greatly reduced from those prevailing when the lines were built. Like "Topsey," our railroad system has "just growed." Less thought has been expended on the location than any other operation of railroad building. But there is some excuse for this, as 30 years ago there were no fixed passenger and freight rates, and a railroad transportation plant was simply a basis for bargaining, success depending on how much could be squeezed out of the public, and not on economical carrying capacity. In locating a railroad in the early days it has been said that they would "follow the grass roots for grade line and the local subsidies for alignment;" that is, they would dig as little as possible and reach any town where the people would subscribe enough money. Railroads so located are left as a bad legacy to those who afterwards have to try to operate them on a paying basis.

There were just as good engineers in the profession then as now; but broad minded managers with visions that could foresee the growth of the country and build accordingly were too frequently lacking. In the report of the Hon. Jefferson Davis, made about 1858, when he was secretary of war, will be found a very good instance of this. It is in regard to the proposed route of the transcontinental railroad as surveyed by General (then Captain) George B McClellan. The board of engineers favored the northern route, following very closely the route, and using the same pass through the Rocky Mountains that the Great Northern Ry. uses today, but this route was condemned by the secretary of war principally for the reason *that there was not enough timber across Dakota and Montana to supply fire wood for the engines and if the northern route was selected the building of the line to the Pacific coast would have to be deferred some 20 years until trees could be planted and a supply of fuel assured*

MAPS SHOWING PRODUCTS AND [illegible] TRAFFIC

To properly locate a railroad one should have about 80 general maps of the country, 30x40 ins. being a good size, so they can afterward be bound in the form of a big flat atlas. They should be specially prepared maps, compiled from all available sources, and printed from a negative on white paper that will take ink and endure usage. Existing railroads and all streams should be strongly featured; the streams especially detailed, to stand out boldly and show the watersheds.

These maps, of course, are duplicates of one another One of them should be colored or diagrammed to show the different densities of rural population as they exist at present and one similarly diagrammed for each period of ten years to come, showing an estimate of the probable population 10, 20, 30 and 40 years ahead. This information will, of course, be an estimate, but it is necessary as railroads are built, more for the future than the present. Information obtained from the various censuses and from the U. S. land office, showing the rates at which the land has been homesteaded or purchased from the government form the best bases for making such estimates.

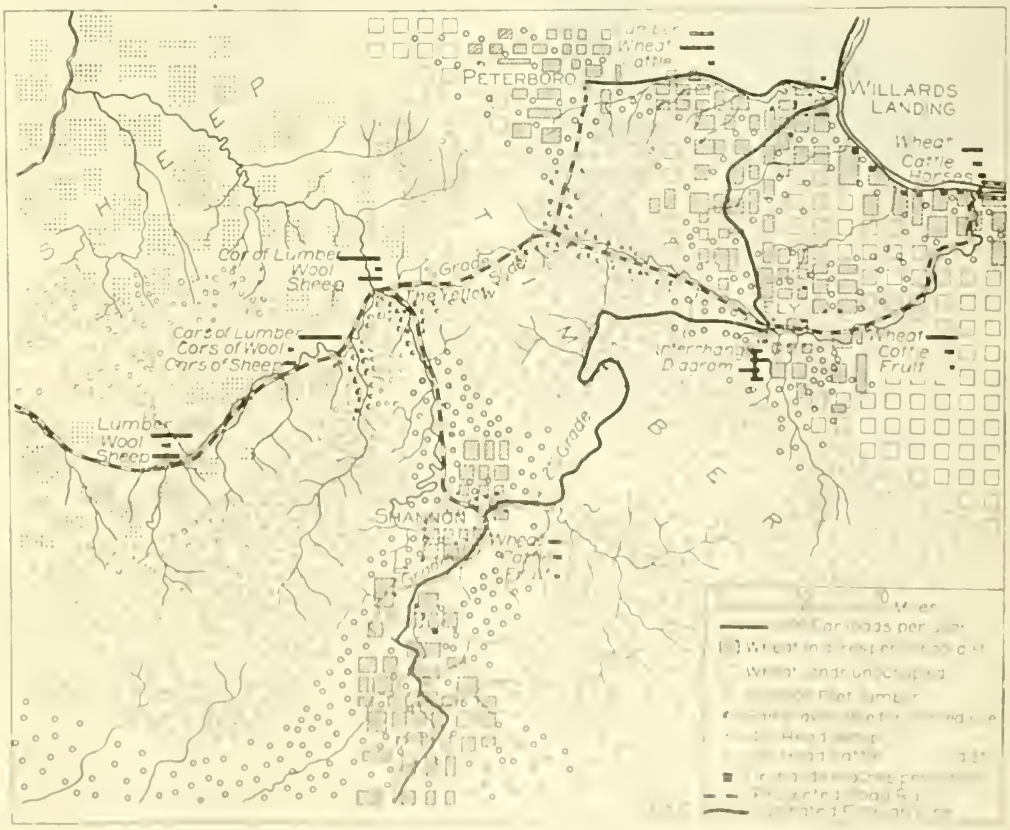


Fig. 1. Map Showing Estimated Traffic from Products and Interchange for First Ten-Year Period After a Projected Railroad Is Constructed.

Then several of these maps should be prepared to show in color and diagram the agricultural products in acreage planted and tonnage produced, and these should all be duplicated for the four future 10-year periods.

Also a set of these maps should be prepared showing the agricultural products in zones of haul on wagon lines to the present existing [illegible] railroad shipping points. Everything that has to be hauled should be looked upon as possible grist to the mill. Overlook nothing, the mineral products, timber, chicken and [illegible] business, and even the probable growth of suburban traffic should be diagrammed in 10-year periods on a set of these maps. One newly built road found itself at a disadvantage the second year of operation, because some of these possible sources of revenue were overlooked. The unexpected growth of the milk business and suburban traffic required radical changes in the location and operation of the road. The increase in traffic even made it necessary to double track portions of the line.

After the above mentioned maps are prepared a great many diagrams and diagrammed maps are made showing the present freight and interchange volumes.

Figure 1 illustrates a portion of a traffic map for the first 10-year period, showing estimated revenue producing products in diagram. In the original these were shown in colors, and a number of conditions were depicted which cannot be reproduced in an uncolored map. The heavy black line is the opposition railroad constructed some 20 years previous. The dashed line is the projection of the proposed new road. Information concerning rivers was obtained from U. S. sectional land surveys. Statistics of agricultural products were secured from township assessors' returns and plat books. As these statistics are given in by the owners they are always sufficiently conservative. It will take one man to each county seat about seven days to obtain this information. Areas of wheat are shown per school district Sheep are indicated by ownership or "bunch." The estimated interchange traffic is shown by diagram at the station where connection would be made with the foreign line. The consideration of the line through this section was abandoned for one further north, owing partly to "railroad politics" and partly to the adverse grade over the "Yellowslide" pass, a very objectionable feature in a trunk line, as it landed it back in the same watershed.

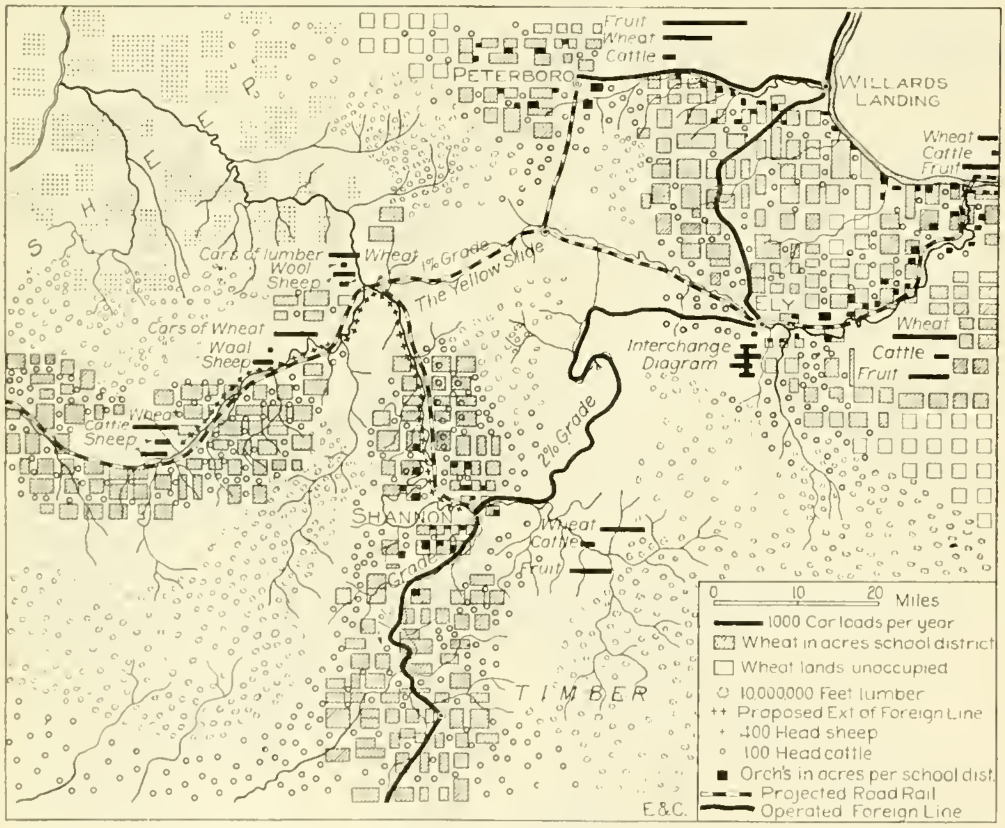


Fig. 2. Map Showing Estimated Traffic from Products and Interchange for Second Ten Year Period After a Projected Railroad Is Constructed.

In Fig 2 is shown a portion of the traffic map for the same section of country for the second 10-year period, showing revenue products in diagram. Note that the timber area has receded, that wheat raising has taken the place of sheep herding in some of the sheep country and that fruits have come in as revenue producers. Also the probability is entertained that the older established road will invade some of the choice territory with a branch line, as shown by the line of black crosses. As a matter of fact, this did occur; that company became active some eight years earlier than was expected, and completely blocked the situation by occupation of the river canyon by a route practically as shown. The estimated interchange traffic for the second 10-year period is shown by diagram as in Fig. 1.

From all this combined information a traffic man and an engineer together can figure out the probable volume of traffic a new line could command. Then the engineer will know how much cost the line will bear and in a general way where to build it to command the most traffic. He is then ready to consider the topography of the country, and begin to plan a line that will haul the products and be so located as to be capable of improvement in grades and alignment when the increase of traffic demands. However, 90 per cent of our roads have not been located in this careful manner, and the result is there are roads over routes where they should never have been built.

Of course there are many exceptions where this systematic investigation is not necessary, such as where a large body of coal or iron is to be the sole revenue producer, or where the road is to parallel an old road for spite work or "railroad politics."

EXPLORATION SURVEY METHODS.

We have now arrived at the point where the engineering work proper begins, and the first proceeding is the exploration. This should be made in a general way, at first covering a large expanse of country; "of an area and not of a line" as Mr. E. H. McHenry expressed it. In the sectionized land portions of the United States the section lines are sufficiently accurate for distance, and the elevations given by the U. S. Geological Survey are a great aid, but in a wild country preliminary survey parties become necessary. It is in this preliminary and exploration work that many a chief engineer has shown himself to be too small-minded to conduct an extensive and comprehensive operation. Too much dependence has been placed on exploration parties consisting of two men, a locating engineer, and a guide sent out to explore the country. While an engineer may not get lost in actuality, he soon loses his connection with sea level datum altitudes, and then he is as good as lost so far as practical results are concerned. No man has a level in his eye, though many think they have, and so it is absolutely indispensable to get a line of levels across the country as soon as possible. The use of hand level and barometer has been argued and the debate fought to a finish with a decision that those instruments have been found lacking. True, a 1 per cent grade line has often been explored out with a hand level, but on a 0.4 or a 0.5 per cent grade a hand level is no good. There are men who can follow a 1. per cent grade, but they are apt to lose sight of a possible 0.5 per cent line that could have been built just as cheaply. Now that the country has been cleared up, and in portions contoured by the U. S. Geological Survey, one can often see where some 40 years ago a misguided engineer trusting to a hand level has deliberately gone up hill to get his beloved 1. per cent line so intently was he following slopes of that degree, and overlooked a far better route but a short distance away.

So the first thing to do is to get a line of levels across the country with a Y-level; without chaining; simply to establish bench marks every few miles. These may not be within 10 miles of the final location, but they will be near enough to be "hooked onto" by the exploration men. One levelman and his rodman will do about four miles a day, so if the contemplated line is somewhere in the neighborhood of 400 miles long it will take about 120 days including Sundays and rainy days for one line across the country. The work of no levelman is sufficiently accurate to be relied upon without its being checked so two more level parties should follow or be on parallel lines with frequent checks between all of the lines. This will insure all the work being on one datum, sea level preferred. So considering the least time it will take to get the field pierced with lines of levels, the Chief Engineer should have out his level parties a year ahead, or while the figuring on the traffic problems is being done. These levelers should be fair-priced men, say at $125 to $150 a month, preferably locating engineers kept on the pay roll for future use. This precision may be unapproved as too expensive by many departmental heads of the school that "just growed," but nearly every one of them has at one time or another been tormented by two or three sets of contours on different datums; trying to "hook up" Smith's line with Jone's line when their contours do not agree. This alone justifies having the whole work on the same datum. Of course there are many cases where a line of levels is not necessary, such as paralleling the water grade of a river or following the shore of the sea.

The U. S. Geological Survey carrys stadia levels six miles per day within an error of 0.5 ft., but this is only attainable with the men and methods they employ; and the average railroad transit man is a poor substitute for them. The Grand Trunk Pacific used vertical arc levels; one survey party from Edmonton east and one from the Prince Albert Branch west, and when they met about half way their levels did not agree within 78 ft. As it was an 0.4 per cent grade one or the other was "in the wrong county" and the work useless. Levels taken by angles generally are worthless unless a real instrument of precision is used, and the speed obtainable with such instruments is too slow. The proper method is raising or lowering the telescope a certain number of distances measured vertically, giving rod readings of 60 or 70 ft., and it is well worth considering by those who want only moderate accuracy and great speed.

Fig. 3 illustrates this method which may be described as follows: 1st position: with the telescope level note that the middle cross wire hits the top of the tree, several feet below which an elevation is to be determined; 1st movement: Bring the top cross wire to the position of middle wire (top of tree), note bottom wire strikes top of fence post near left end of wire. Keep eye on post; 2nd movement: Bring top wire to position of bottom wire (top of fence post) and note that bottom wire now strikes a clod in field. Keep eye on clod; 3rd movement: Bring top wire to position of bottom wire. Bottom wire now reads

11.5 ft. on rod. 4th movement: Bring both wires down to see how much is intercepted on rod between top and bottom wires, in this case 8.3 ft. which is the height of each step down. (3×8.3)+11.5=36.4 ft.=total height from top of tree to surface of ground (or top of stake) where rod was held. Elevation of top of tree = elevation of telescope = H. I. H. I. — 36.4 ft. = desired elevation at point where rod was held.

But this cannot be done against a sky line; the instrumentman must have points on the earth to keep track of his steps downward or upward. As previously explained he goes down from the top of a tree (seen through the telescope) to the top of a fence post, and from the top of a fence post to the top of a clod on the side of a hill, and from the top of the clod to an intersection on the rod. It is extremely simple to see it done, but the man should have a practiced eye and not lose sight of his tree, fence post, and clod. The Geological Survey has some men very expert in the use of this method, and there are also alidade telescopes that have a computer on the side in case the topographer is city bred, and has not an eye for trees, fence posts and clods.

I have described these methods in detail with the hope that more reliable and fuller information may be obtained through their use than is usually brought in by the average two-man exploring party—those hand-level and aneroid outfits. As a general thing the chief engineer waits impatiently for some results from them, while the precious summer months slide by, and at last he receives a disgraceful scrap of a map, the river courses generally well defined (the explorer's idea of them) but for the rest there are only such notations as "High ground here," "Low ground," "Swampy," "Full of pot holes," etc., camp sites and places of lodgement receiving equal consideration. What the head office wants is contours with a sea level datum. The map in hand might represent Jones' idea of where the line should run (and where Jones slept and ate), but it is a hard job to "hook it up" with Smith's map and line, or with that of the next man above or below him. I am in favor of the small plane table with stadia wires in the telescope of the alidade, the party to consist of the topographer, rodman, and teamster-cook. In a settled country the latter can be dispensed with, but the topographer and rodman should each have a horse and buggy. Those who have not seen a survey party of this kind at work have no idea of the rapidity with which it can cover the country. The line run is a stadia traverse. No points of control are necessary, and the levels can be corrected on the bench marks set the year previous. If the topographer is a locating engineer the line will follow some sort of a possible railroad route, and being in light running order he can try very many different ones. A profile can be kept up, using standard A paper mounted on a double roller exploration kit. The topographer and his rodman must of course know the business, agree on a set of signals, and work in harmony. There are of course many "quirks" in the business. The drawing should be protected by two or three extra sheets of common paper, well thumb-tacked down, to keep it from soiling and rain drops. To get at the main sheet cut away some of the covering exposing as much as will likely be plotted in one day. On a prairie country an outfit of this kind will do 10 miles a day, contouring the country on either side as far as the eye can see. In a timber country it hits for the hilltops. In an open country it goes up the hollows, and the results in already-plotted contour map and profile are far better than the old style "Low ground," "High ground," "Pot holes!" maps.

With one of these topographical exploring parties to about every 40 miles of line, the chief engineer can begin to get in reliable data in the first three months of the second season after the inception of the project. In a wooded unsectionized country axmen may be needed but generally the timber can be wiggled through by following paths or up watercourses without much cutting, but it is a hard job for the men on a party and no place for a novice.

USE OF EXPLORATION AND TRAFFIC MAPS FOR PAPER LOCATION

By September of the second year the chief engineer will have a pretty complete map of the country,—a number of maps showing the detail and one general map with 50 ft. contours. On this latter map should also be shown the towns in diagram illustrating their present and future traffic values, to determine whether it is worth a slight deviation in line to reach any of them that may be left to one side by the best topographical location. Too often this question has been decided at a champagne dinner, by a meeting of influential citizens with the president of the road.

It is in the general selection of the route that the engineer can show the most skill and foresight. It is for this purpose that extended contour topography is strongly recommended so that the chief engineer can grasp the whole situation. If the line is long and the country extensive it will be impossible for him to see it all except on paper, and this paper should show him something mathematically reliable to relieve him from depending on the ideas, advice, and prejudices of his subordinates. A well-selected line if necessary, can at first be constructed with many stiff grades, short ups and downs, which can afterwards be remedied with a steam shovel; but a poorly selected general route, incapable of being remedied, will sooner or later be found to ill serve the country through which it runs and the investors who built it. In the last ten years miles of old line have been thrown away and replaced by new line often miles away from the original road. Even this is often impossible on account of the towns that have grown up since the road was built and which the railroad commission will not allow the railroad to abandon.

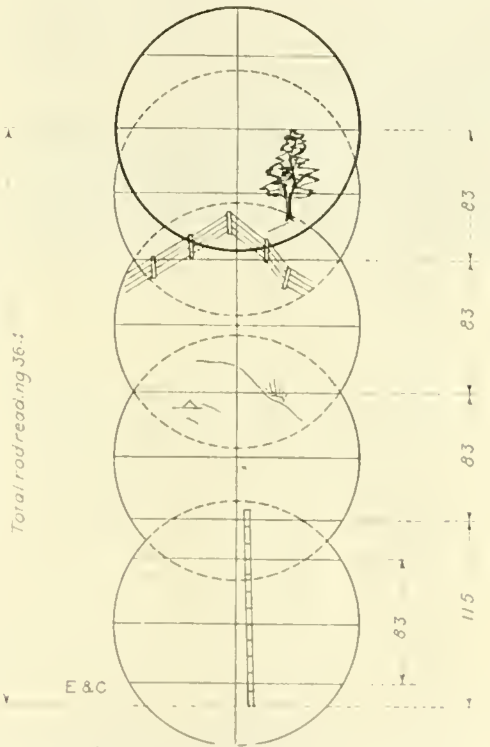


Fig. 3. Illustrating Method of Using Stadia Wires for Obtaining Elevations on Long Sights.

The assembling of the data brought in by the plane table exploration parties is somewhat of a task, but the results are far better than relying on Smith and Jones and their aneroids. Besides the district maps detailed on large scale, and the small scale general map, a plaster cast can be made of the whole territory to a grossly distorted scale. The water shed divides will appear like mountains and the valleys like canyons. With an object lesson like this at hand it is easier to talk to the board of directors, and it becomes a pleasure to meet a delegation of influential citizens who want the new railroad to come to their town. Contour maps mean nothing to them, but from a plaster model they can see that their town is too high or too low, and depart without feeling any grudge against the new enterprise.

CALCULATION OF HAULING CAPACITY.

In constructing any plant, any factory, any irrigation ditch, any city water works system, any electric power plant, its capacity, the capacity of the plant, the amount it will deliver, is the first thing thought of. But most of the railroads in America were built without any thought of their hauling capacity, and a number of them are yet operated without the managements knowing whether their roads are working reasonably near to their tonnage limits or not. The falling short is for two reasons, namely: (1) The inability of the traffic departments to provide the tonnage owing to competition, or (2) The inability of the operating departments to handle the tonnage. The latter may be owing to poor track, lack of terminals, lack of motive power, or to poor location of routes over which they run their trains. Often a random guess is made and the motive power is increased, with the hope of curing the ill, but quite frequently the new motive power and the location of division terminals is wholly out of harmony with the grades and alignment over which the trains are operated. Thanks to some of the brainy men of the profession we now have formulas for engine tonnage diagrams speed profiles, train sheet diagrams, tonnage profiles, terminal inter change diagrams, etc., etc., so that the capacity of a railroad as a plant can be figured out, and the proper remedy applied.

This knowledge is indispensable to the up-to-date locating engineer. He must know the probable amount of tonnage to be hauled, and build the plant of the proper capacity to transport it at a profit, and so located that it can be improved to take care of the increase of traffic in the future. In former days we used to hear a locating engineer speak proudly of the fact that he "got a line through," but never a bit did he speak of the hauling capacity of the plant if built on that alignment. Most of our big railroad systems are built 30 miles on a line located by Jones, the next 10 miles on Smith's line, the next 35 on Anderson's line, and the next 72 on Cumming's line; lines often individually good, and representing the instructions of the chief engineer as to maximum grades and curvature, yet having different carrying capacities, and totally out of harmony with the country as a whole, and inadequate to transport its products at a profit.

With the details of a preliminary survey and projection and final location this article does not deal. The elements of such work can be found in books and the practical experience is common to many. But the necessity of the chief engineer having an extensive small scale topographic map, and being able to get a broad view of the general situation is a point that we endeavor to emphasize. Those who have seen the reconstruction of some of our cross country located trunk lines, know of the millions of dollars that have been spent to rectify the mistakes of those who have gone before. Depending as they did on the individual judgment of some of the locating engineers, no one of the men at the head of affairs seems to have seen the sticking points that should have been avoided or reduced, and were satisfied with a so-much-a-mile railroad in the expense of construction, and a low average railroad in capacity of transportation.

Engine tonnage diagrams, tonnage profiles, and speed profiles have all been dealt with in technical articles before, and will not be taken up here, but they are nevertheless indispensable in modern location work. After the projection profile is completed, a tonnage profile should be made of it. Then from the engine diagrams a speed profile for each class of train can be made, and a theoretical schedule board prepared. These should be duplicated in ten year periods for the increased tonnage. In other words work the trains on paper. They will have to be worked in actuality afterwards, and it is well to know what can be done before millions of dollars are [illegible] the

the line. The train diagramming and tonnage figuring should be done on the projection profile made from the preliminary survey. Field location costs so much money, sometimes $100 a mile and more, and it may be useless to expend that much before the working of the road is tried on paper. One of the first things discovered will be that the passing tracks are not situated to the best advantage, and the shifting can be arranged for accordingly. It is cheaper to find it out on paper than it will be for the superintendent to rearrange them on the ground 10 years later. This information from the ideal speed profiles and train diagrams will also aid in the selection of water stations where the country is not arid and there is any choice; and will also strengthen the chief engineer to put his foot down on some townsite schemes. These would be towns located at points where it would be a great disadvantage for trains to stop, to meet and pass. On one very expensive extension built not many years ago, mostly a cross country location consisting of alternating tunnels and trestles, there was not room for enough passing tracks to accommodate a tonnage that would pay 4 per cent on the investment. The whole road went into the hands of a receiver. In these criticisms, railroad etiquette forbids the mentioning of names, but some operating officials will well know what roads were in mind when these remarks were written.

When it is said that a speed profile should be made after the projection is completed, it is of course contemplated that a regular preliminary line with its accompanying 10 ft. contours has been run over what seemed to be the ideal route. This route being wisely selected mathematically and logically from the plane table exploration maps, combined with the maps showing products and traffic, will save many an expensive full-corps preliminary line; lines that we used to depend on so much for knowledge that we should grasp in a grander and more comprehensive way. We used to "feel the country out": miles on miles of expensive chopping through the woods, and in the end arrive at a jumping off place. It is high time this method was dropped, but many will stick to it, because that is their caliber.

CONCLUDING REMARKS.

It is not the intention in this article to go into the details of the actual location survey. The adjusting of alignment and grades to topography is a well known science, but I am sorry to say that there is a great variation of personal aptitude in the profession. Some engineers are born locators; others are not, and still have not been kicked out. As a general thing the backwoods and poverty in early youth have produced the best locating engineers. There is one such engineer who was raised a cowboy and knew not even arithmetic until he was 24 years old, when he got a job as "mule skinner" on a survey party. Today he is one of the most investigating accomplished mathematicians in the profession. Some locators have such marked characteristic technique in their work, that from the way the alignment of a line is adjusted to a contoured valley, another engineer who knows them can tell who did the work even with the names of rivers, towns and mountains omitted on the map. Their work is as characteristic of the individual as that of an artist in oils and brushes. The style of one in particular, who was a chief engineer then and is now, was to throw the line into the hill, throw it in a little more than anyone else would have done, more than some of his locators had it. This became so well depended on by a small group of contractors that they would arrange their bids accordingly, and made comfortable fortunes oft the slides that came down. That it was wise to throw it in is still a disputed question, for if it had been kept out it might [illegible] been washed away and traffic interrupt- [illegible] cost of which some of the locators [illegible] take into consideration, and "J F." [illegible] ed is now [illegible] of the most advanced [illegible] his time.

[illegible] locating engineer to try the [illegible] ays do better than the first [illegible] aps and profiles prepared [illegible] r the maps and profiles do, and should, represent all the first one knows. They represent hours of study and thought and the accepted result out of miles of unsuccessful experiments. They are the concentrated form of all this effort, and they tell the tale plainly. The second engineer will have the advantage of all this knowledge and have his own knowledge of the line besides. In most cases he will see something that the first one overlooked owing to some handicap of the work, and be able to place a better alignment on practically the same contours. How far they can keep on getting a better line, whether there is any better result from having third and fourth engineers, is still a disputed question, but there should be at least two to look over the maps, with the chief engineer as the final arbiter. The handicap on the first one may be owing to a one-sided view, to the weather, to the hardships of the trail, or just naturally to his being lost in the woods, but it is no test of his ability to have another one suggest something better after he has shown all he knows. Perhaps number two if he took number one's place could not do nearly so well in the raw country.

No one man has a patent to this knowledge; it is the combined work of many. The railroad engineering profession is indebted to J. D. Isaacs for locomotive diagrams and speed profiles; to George Kopp for industrial maps and traffic diagrams; to A. Stotlar for operating cost sheets; to Basil Duke for rapid topography methods. Even with them these specialties may not have been original, but they were among the first to use them where they could be seen by others.

Most of these scientific and mathematical methods of solving railroad engineering problems are of great value for use in investigating and improving the grades, alignment, and motive power of existing operated roads. General managers should know the diagrammed capacity of their motive power in relation to the grades over which it is operated. It has been common practice for yardmasters to take off 10 per cent from a train-load in making up, then the dispatcher takes off 10 per cent, and out on the line the crew sets off 10 per cent on a fictitious "got stuck." So if the ideal working capacity is not known, no club is held over the heads of the men, and the engines are run short of what they ought to be pulling. Some may say that all this mapping, diagramming and figuring is of no use; that all 0.5 per cent or all 1.0 per cent lines have the same capacity. True, they would if all the country was on a level, or on an even slope for hundreds of miles, instead of being undulating as it acually is. But God did not make the earth that way. The engineer's business is to adapt the lines of travel to it.

Cost of Location Survey for a Short Railway Line.

(Contributed.)

Location surveys for a railroad were made in 1912, in one of the eastern states, during which accurate records were kept for the purpose of producing authentic cost data. It is believed that the accompanying tables will prove interesting and instructive.

The survey was for a branch 30.75 miles long, from an already established line to a manufacturing city; with a branch survey, to another city, 12 miles in length. Topographical considerations, aside from large bodies of water and high hills, were found to be of minor importance, the chief concern being so to approach the intersected streets that grade crossings might be cheaply avoided. Most of the distance was through a light growth of timber, requiring much chopping and trimming. No topography was taken, but all streets and water courses along the line were carefully surveyed and the boundaries of private lands were run out.

For three weeks the men were boarded in hotels in the two terminal cities; during nine weeks they were carried by team or railroad to the work from headquarters on the main line; and for twenty weeks and two days they lived in camp. From the camp, which occupied three different sites, about 26 miles of location were made, teams being kept with the camp for transportation.

TABLE I.—DISTRIBUTION OF LABOR.

Description of work—	Cost.	Cost per mile.	Pct. of labor total.	Pct. of grand total.
Running the line finally adopted	$1,500	$35.10	39.7	20.8
Running lines afterwards abandoned	432	10.10	11.4	6.0
Surveys of intersected streets	666	15.60	17.6	9.3
Leveling on line finally adopted	156	3.65	4.1	2.2
Leveling on lines afterward abandoned	65	1.52	1.7	0.9
Leveling on intersected streets	20	0.47	0.5	0.3
Meandering ponds and streams	57	1.33	1.5	0.8
Surveying private boundaries	382	8.95	10.0	5.3
Triangulation and traverse lines	146	3.42	3.9	2.0
Exploration	10	0.23	0.3	0.15
Check levels	10	0.23	0.3	0.15
Office work by field men	61	1.43	1.6	0.8
Holidays, absences and rainy days	279	6.55	7.4	3.8
Totals	$3,784	$88.58	100.0	52.50

Table I of labor distribution does not include any general officers' salaries nor that of the chief engineer. The map drawing was done in the general office and does not figure in these tabulations.

Surveying instruments were supplied from those previously in use by the company, and interest on their cost is charged under "Field and Office Equipment." The camp equipment, consisting of seven tents, complete mess outfit, cot beds, blankets and quilts, was purchased second-hand at a discount of 50 per cent.

TABLE II.—DISTRIBUTION OF EXPENSES.

	Cost.	Cost per mile.	Pct. of exp. total.	Pct. of grand total.
Field and office equipment	$ 160.49	$ 3.75	4.7	2.2
Railroad and street car fares and expenses	453.73	10.60	13.3	6.4
Board and lodging	167.00	3.90	4.9	2.3
Team transportation	836.32	19.50	24.6	11.6
Camp equipment	209.16	4.90	6.2	2.9
Camp maintenance	1,488.92	34.82	43.9	20.7
Purchased information	81.38	1.90	2.4	1.4
Mail, telegraph and telephone	5.56	0.13	...	...
Totals	$3,402.56	$79.50	100.0	47.50
Grand totals	$7,186.56	$168.08	.....	100.00

During the nine weeks above mentioned many days were lost on account of rain for

TABLE III.—ANALYSIS OF CAMP MAINTENANCE.

Commodity—	Cost.	Cost per meal—4,047 meals.	Cost per man per day.	Pct. of total cost.
Meat, fresh and canned	$ 373.72	$0.092	$0.239	25.1
Fish, fresh and canned	51.93	0.013	0.033	3.5
Potatoes	29.84	0.007	0.019	2.0
Other vegetables	53.93	0.013	0.034	3.6
Fruit, fresh and canned	120.58	0.030	0.077	8.1
Groceries	108.56	0.027	0.069	7.3
Flour	31.92	0.008	0.020	2.1
Eggs	59.39	0.015	0.038	4.0
Sugar	27.49	0.007	0.018	1.9
Coffee	26.50	0.006	0.017	1.8
Tea	9.06	0.002	0.006	0.6
Butter	37.27	0.009	0.024	2.5
Milk	73.03	0.018	0.047	4.9
Ice	18.61	0.005	0.012	1.3
Fuel	36.23	0.009	0.023	2.4
Light	6.50	0.002	0.004	0.4
Water—6 weeks from city main	5.00	0.001	0.003	0.3
Land rentals	10.00	0.002	0.006	0.7
Cook's wages	305.34	0.076	0.196	20.5
Labor moving camp	104.02	0.026	0.067	7.0
Totals	$1,488.92	$0.368	$0.952	100.0

which the men, being at home, were not paid. The pay of the party was as follows:

Position—	Pay per day.
Assistant engineer in charge	$4.50
Transitman	3.33
Leveler	2.50
Axman and teamster, 7 days per week	2.25
Chainmen	2.00
Rodman	1.75
Axemen	1.50
Cook, per week	15.00

A study of the tables does not suggest much resemblance to similar ones previously published, the most marked difference appearing in the matter of camp maintenance, Table II. Much of the higher cost shown here is doubtless due to the high cost of living now prevailing. Also much may be due to the fact that all the men were accustomed to a pretty good table and an effort was made to provide them with home comforts.

The total cost per mile given, $168.08, Table II is the cost per mile of final location, and will be seen to include the cost of preliminary lines, and all the detail surveying necessary for complete land plans.

Methods of Concrete Construction in Track Elevation Work at Cleveland, Ohio.

The track elevation of the New York, Chicago & St. Louis R. R. for some five miles in Cleveland, O., called for about 63,000 cu. yds. of concrete work in street bridge abutments and retaining walls. Some of the methods of concrete construction are described by Mr. A. J. Himes, engineer of grade elimination, in a recent bulletin of the American Railway Engineering Association, and of these we publish here the ones of much general interest.

Three concrete mixers were used; they were mounted on cars. Two were equipped with hoisting engines for charging. The third was charged by wheelbarrows moving over the cars, and was used in building the floor slabs and other work where a large output was not required. The manner of operation is illustrated by Figs. 1 and 2. In Fig. 1, the housing was carried over the boiler and the incline proved too steep for economical wheeling. The method was at once abandoned for that of Fig. 2. At a later date wheeling was again resorted to, but the charging of the mixer was done at a lower elevation and with better results.

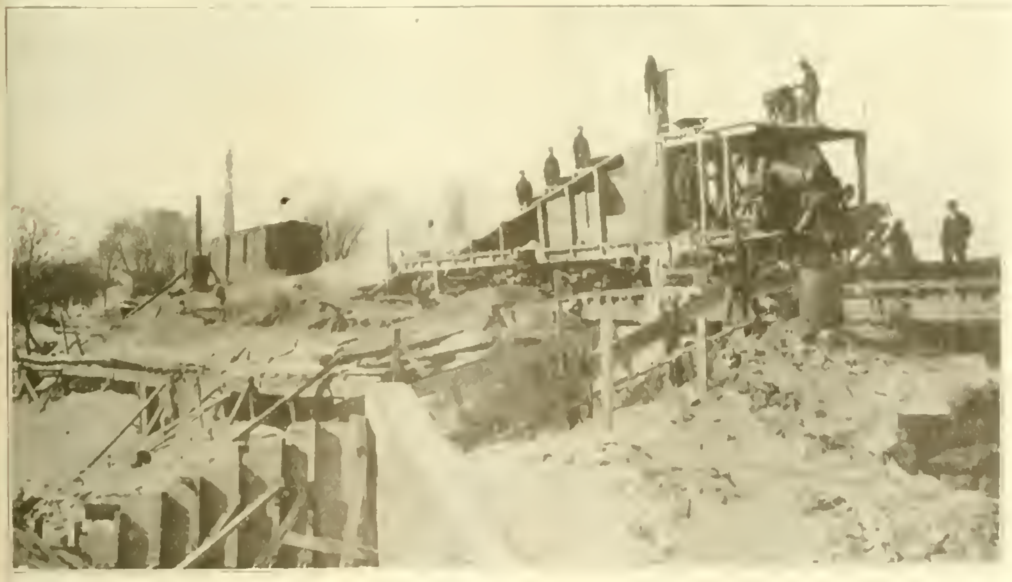


Fig. 1. First Arrangement of Car Mounted Mixing Plant Abandoned Because of Too Steep Incline.

Between Lakeview road and [illegible] St., a distance of 2,600 ft., the streets—[illegible] of them—were so close together and the available right-of-way was so narrow that it was thought best to make use of a temporary trestle and to mix the concrete on the trestle and pour it by chutes into the abutment forms below. One reason for so mounting the mixers was that because of the large number of structures and their small average volume, much time could be saved in moving the plant. Furthermore, there was no room for derricks, stock piles and stationary plant.

The concrete for many footing courses and small concrete structures was mixed and placed by hand. This was partly because the small yardage made it economical, partly to hasten the work when the mixers were busy and in the case of the footings, when sheeting and bracing were used in the foundations, it was better to get the footings in, part or all of the bracing removed and the forms erected before bringing the mixer on the work; otherwise it would stand idle a part of the time and thus reduce the volume of concrete placed per month below the rate necessary to keep up with the schedule. The maximum output of one mixer per day of 20 hours was [illegible] cu. yds. The average output of one mixer per month for 11 months was 1,315 cu. yds.

All structures were divided into sections in such a way as to preclude cracks from expansion and settlement, and each section was poured complete in one run. The concrete was generally mixed quite wet, the mixture most used being 1:3:6, in bridge floors and reinforced work it was 1:2:4. The wet mixture was necessary in order to have it flow well in the chutes, and to insure a dense water tight product without tamping. It was aimed to drive the surplus water out from the concrete at the rear of the masonry and thus [illegible] of laitance on the face of the structure. In this respect the success was very good, but it might have been better.

Where the mixer car was obliged to stand upon the ground towers, Fig. 3, were frequently used for elevating the wet concrete to a point where it might be distributed by gravity. In some cases this method was very economical. At Euclid Ave., after railroad traffic had been turned over the bridge, there was so much traffic above and below and so little available space that no other method seemed at all comparable in either economy or speed.

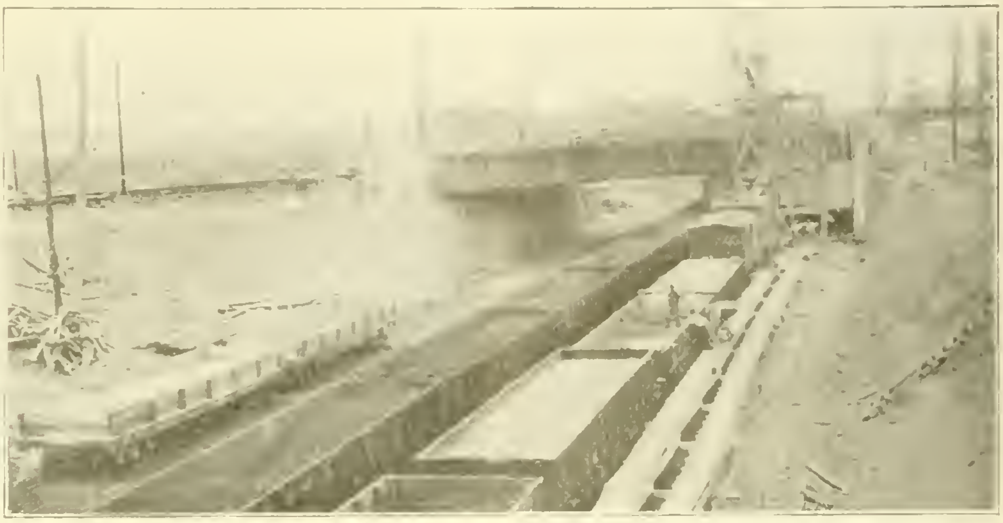


Fig. 2. Mixing Plant with Charging Car Truck Alongside Material Cars and Tower for Distributing Concrete.

Wooden chutes were made 2 ft. wide and 8 ins. to 10 ins. deep. With planed boards the preferable slope is 1 ins. per foot. Slopes of 2 ins. per foot were used at times, but a man was then required to keep the chutes clear. A slope of 6 ins. per foot will cause the ingredients to separate and requires the use of baffles to retard the motion. Iron chutes were made 20 ins. wide and 8 ins. deep. The maximum slope used was 4 ins. per foot, the minimum slope 2 ins. per foot. The slope is, of course, dependent upon the amount of water in the concrete.

Great pains were taken to remove all form marks and other defects from the surfaces of the masonry. The cheapest method and one that proved generally pleasing was to bush-hammer the surfaces. Such work was done at a cost of 1 cts. per square foot. Other surfaces were rubbed smooth with carborundum bricks at a cost varying from 1 cts. to 10 cts. per square foot.

Little difficulty was experienced in making repairs or correcting defects. Surfaces to be patched were carefully cleaned and then soaked with water. The mortar was applied in comparatively thin layers by throwing on forcibly with a trowel, each layer being permitted in turn to harden. Such work has gone through three winters without sign of failure.

Iron trowels were never used in finishing the surface. Very smooth surfaces were secured with carborundum. Rough, sandy surfaces, resembling Cleveland sandstone, were secured by rubbing with wooden floats while the concrete was still green. This is an excellent finish and can be done at a cost less than bush hammering if there are no surface defects to be removed.

When the aggregate was desired to show, the mortar was brushed away while green with wire brushes. A good finish of the latter sort requires the aggregate to be uniform in size and uniformly placed. It is not easy to secure. A very beautiful surface may be secured by bush hammering a concrete made with quartz gravel.

The concrete work was carried on continuously the year round. In freezing weather, the water and sand were heated and salt water (a saturated solution) was used when there was to be [illegible] in contact with the concrete. In but one instance was there any sign of a failure. This case was in a section of a retaining wall that was placed by hand. The final conclusion as to the cause was that salt had been thrown in the mortar without being first dissolved in water.

Washing Sand.—Any sand for use in concrete, showing signs of vegetable matter, mud or other impurities, should be thoroughly washed. The following method will be found useful: Build a platform about 12 ft. long by 4 ft. wide, with one end about a foot higher than the other. On the lower end and sides nail 2x6 in. boards to hold the sand. Spread the sand over the platform in a layer 3 or 4 ins. thick and wash it with a hose. The washing should be started at the upper end, letting the water run through the sand and out over the board at the bottom. All of the sand should be thoroughly agitated during the washing so as to allow the water to carry off the dirt. After washing, the sand should be piled up where it will not come in contact with the dirty water. Drain this water off to some suitable outlet and do not use it for mixing the concrete.

The Principal Long Tunnels of the World.

The following table of the principal tunnels of 2 miles or more in length has been compiled by the "Engineer" of London. There are 52 tunnels listed, of which only one is in the United States, which is accounted for obviously by the fact that the list includes only railway tunnels and also only tunnels wholly in rock. Long tunnels now being constructed outside of Europe, even those of the class specified, are also not included, which eliminates other possible representation, as for example the Montreal Tunnel, by works of the American continent:

TUNNELS IN EUROPE OF TWO MILES IN LENGTH.

Name of tunnel—	Country.	Length. mi.	yds.	Summit level, ft.	for traffic.
Simplon*	Switzerland-Italy	12	458	2,313	1906
St. Gothard	Switzerland-Italy	9	564	3,788	1882
Löschberg	Switzerland	9	55	4,077	1913
Mont Cenis	France-Italy	7	1,730	4,248	1871
Arlberg	Austria	6	404	4,300	1885
Ricken*	Switzerland	5	610	650	1910
Tauern	Austria	5	546	4,020	1909
Ronco	Italy	5	277	.....	1888
Tenda*	Italy	5	56	3,260	1899
Karawanken	Austria	4	1,683	2,088	1906
Jungfrau*†	Switzerland	4	834	11,220	1912
Borgallo	Italy	4	700	.....	1887
Severn	England-Wales	4	636	.....	1886
Turchino*	Italy	4	10	.....	1900
Wochelner	Austria	3	1,647	1,761	1909
Albula*†	Switzerland	3	1,150	6,133	1903
Totley	England	3	950	.....	1893
Peloritana*	Sicily	3	686	.....	1885
Gravehals*	Norway	3	516	2,844	1909
Standedge	England	3	60	.....	1850
Woodhead	England	3	13	.....	1845
Bosruck*	Austria	2	1,693	2,405	1906
La Nertho	France	2	1,581	.....	....
Kaiser Wilhelm	Germany	2	1,069	.....	1879
Echarmeaux	France	2	1,020	....	1895
Blaisy	France	2	990	.....	....
Sodbury	England	2	913	.....	1903
Credo	France	2	800	.....	....
Vizzavona*†	Corsica	2	778	2,971	1889
Disley	England	2	346	.....	1902
Col de St. Michel*†	France	2	292	.....	1901
Bramhope	England	2	234	.....	1849
Festiniog*	Wales	2	206	.....	1879
Cowburn	England	2	182	...	1893
Meudon	France	2	134	.....	1900
Giovi	Italy	2	45	1,890	....
Col des Loges*	Switzerland	2		3,200	....
Cremolino*	Italy	1	1,718	.....	....
Calrasca*	Italy	1	1,520	.....	1906
Hauenstein	Switzerland	1	1,210	.....	...
Semmering	Austria	0	1,333	2,910	1850

THE LONGEST TUNNELS OUT OF EUROPE.

Name of tunnel—	Country	Length mi.	yds.	Summit level, ft.	Opened for traffic.
Hoosac	United States	4	685	...	1876
Khojak‡	Baluchistan	2	769	....	1892
Suram	Caucasus	2	750	.....	1895
Caldera*	Peru	1	1,320	15,771	1893
Beacon Hill*	China	1	641	.....	1910
Transandine*†	Chile-Argentine	1	551	10,500	1911

LONG EUROPEAN TUNNELS NOW BEING CONSTRUCTED.

Name of tunnel—	Country	Length. mi.	yds.	Summit level, ft.	Opened for traffic.
Hauenstein Base	Switzerland	5	0	.....	....
Somport*	France-Spain	1	1,512	.....	....
Grenchenberg	Switzerland	1	0	1,787	...
Puymorens*	France-Spain	3	317	.....	...
Mont d'Or	France-Switzerland	3	1,395	.....	....

*Single line tunnel. †One metre gage railway. ‡Standard gage of India.

Use of Metal Railway Ties in Switzerland.—The scarcity of wood for general purposes and the increasing cost of wooden railway ties in Europe has encouraged the manufacture and general use of metal ties in recent years in the railways on the Continent, according to a consular report. About 70 per cent of the ties in use on the Federal railways of Switzerland are metal. The majority of the wooden ties used are made of Swiss wood.

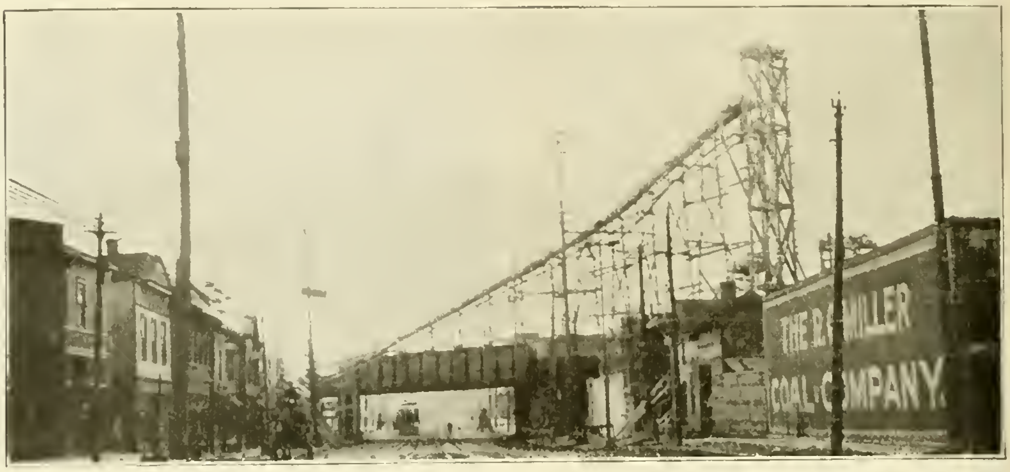


Fig. 3. Tower and Concrete Chuting System for Euclid Avenue Bridge.

Metal ties were first used on the Swiss roads in 1881, since which time they have gradually replaced, to a large extent, the wooden ties. Iron ties are especially recommended on the Swiss roads where a good quality of gravel is available for ballast. Wooden ties are used in tunnels and in localities where the conditions do not permit of good drainage along the railways. They are also generally used in street crossings in towns and cities where the track is graded with gravel up to the level of the ties.

While the cost of metal is greatly in excess of that of wooden ties, the longer duration of the former makes them cheaper in the end, as they do not have to be replaced so frequently, which is an expense that has to be taken into consideration when wooden ties are used in road construction.

The following prices are paid by the Swiss Government for plain, or unoiled, wooden ties, 9 ft. long, 10 ins. wide, and 6 ins. thick: Oak, $1.25 to $1.50; beech, $0.81 to $1,12; larch, $0.83 to $1.16; pine, $0.73 to $0.96. Oiled wood ties of same size and quality: Oak, $1.58 to $1.87; beech, $1.45 to $1.81; larch, $1.14 to $1.54; pine, $1.04 to $1.35.

Iron ties 9 ft. long, 9¼ ins. wide, by 5¼ ins. thick, weighing 160 lbs., sell for $2.30. The metal ties used on the Swiss railways are imported from Germany and are purchased on contract by the Federal Government.

Graduate School of Engineering at Columbia University.—When the School of Mines of Columbia University celebrates its fiftieth anniversary in May next, the end of the old curriculum will be marked, for with the beginning of the next scholastic year, in September, will come the consummation of a plan conceived by the founders of the school, half a century ago—a graduate school of engineering—on the same footing as the other graduate schools of the University. Under the new plan three years of specialized study in the various branches of engineering will be offered leading to the several technical degrees in the School of Mines, Engineering and Chemistry. The requirements for admission to these schools will be the completion of at least three years of a college or scientific school course, including thorough instruction in mathematics, chemistry and physics; and in addition certain courses in drafting, shopwork, surveying and mineralogy.

To prepare for this graduate course inside the university a new course has been prepared in Columbia College leading to the degree of Bachelor of Science, in which a little more than one-half of the time of the student is given to the requirements, and the remainder is devoted to English, the modern languages, histroy, philosophy and the political sciences.

The Suez Canal authorities announce that the maximum draft of water authorized has been increased by 1 foot, making it 29 feet. Six years ago the depth was increased to 28 feet.

Engineering and Contracting

Devoted to the Economics of Civil Engineering Design
and to Methods and Cost of Construction

Volume XLI. CHICAGO, ILL., FEBRUARY 25, 1914. Number 8.

The Technical Results of the Conference in Concrete Road Building.

The business of concrete road building is asserted to have received a notable impetus from the conference held a fortnight ago in connection with the Chicago Cement Show. One object of the conference was therefore attained. To the interests which promoted the conference and by whom it was financed this object was perhaps the first. They are therefore, to be congratulated. But we believe that these interests had also another object which was the technical advance of concrete road building, and which was certainly the object to seek which most of the engineers and highway officials in attendance came to the conference. A question then which concerns all who created the occasion and all who participated in its activities is: What were the technical results of the conference on concrete road building?

One result of very considerable technical importance was the enlarging of the field of vision of many of those in attendance. Concrete road building has been chiefly promoted by special interests. In this respect the conditions are no different than they are for brick or bituminous or wood block roads. Neither is there in this fact any cause for censure. But concrete is a newer material to be specially promoted for roads and its actual use has been in places more widely separated than is the fact with brick or bitumen or wood block. Also much of the literature on concrete roads, and certainly the literature that has been most widely disseminated has been literature inspired by business developing rather than scientific considerations. All this has acted to keep provincial the knowledge of concrete roads. A great many road builders have known actually of concrete roads only the facts developed by the roads in their own locality; of the facts regarding other concrete roads they have known only those which the agencies interested in the business of concrete road building deemed most wise to emphasize. The conference enlarged the vision of these men by its more complete disclosure to them of all facts. The good results of this were very apparent as the conference progressed, encomium and censoriousness both moderated toward scientific judgment. This is the mental attitude which will do most to develop the real merits of concrete as a road building material.

A review of the technical committee reports as abstracted in this issue and that of a week ago discloses only here and there any novelty in thought. They are distinguished by [illegible] conservatism. This is also true of the rules of recommended practice adopted by the conference. Tradition is adhered to in design and in construction methods and materials in subgrade construction, expansion joints and mixing, placing and curing the concrete. This is wholly natural and proper and it lends emphasis to the few new thoughts [illegible] attraction for the more expert.

Engineers will agree, we believe, that the one report most fruitful in new ideas was that of the committee on contraction and expansion of concrete in concrete roads. Based on test data this report makes the important assertion that alternate wetting and drying of a concrete road slab produce contraction and expansion stresses and movements very much greater than do alternate rise and fall of temperature. Stated spe[illegible] rise in temperature [illegible] concrete road slab will [illegible] the concrete cause contraction which may easily exceed in intensity and magnitude the expansion due to the increased temperature. The committee report which we publish in this issue presents proof of this phenomenon which perhaps is not conclusive but which is strong enough to demand consideration.

A less clearly demonstrated and equally novel assertion of this committee report is that a dished sub-grade tends to reduce longitudinal cracking of the concrete slab, slab thickness being equal, as compared with crowned or flat sub-grades. The statement in the report is as follows:

> A concrete road laid on a crowned sub-base would be the most liable to crack longitudinally and on a dished sub-base the least liable to crack. The resistance offered to contraction of a slab with a crowned sub-base is greater than that offered by either a dished or flat sub-base, because the concrete must move against the effect of gravity. With the dished sub-base the concrete contracts with the effect of gravity and therefore is stressed the least.

Whatever the practical value of dishing the sub-grade may be, the idea enunciated by the report is interesting. A road slab built on a sub-grade having a longitudinal incline tends to slide down grade; sometimes slabs so located have actually moved down the hill. With a dished sub-grade, then, gravity tends to slide both sides of the road slab toward the slab center and thus tends to counteract any stress which acts to separate the slab, or to crack it, at the center.

Reports and discussions of construction methods were the ones most lacking in the work of the conference. Cost of concrete roads is a prime factor in determining the preference of concrete construction to other kinds of road construction. Cost of concrete road construction is very largely labor and plant cost—hauling, preparing sub grade, concrete mixing, placing and curing, and these costs are determined primarily by the quality of construction work, management—plant selection, organization, construction methods, etc. Too few data were presented by the committee reports on matters of these kinds and the few data presented were in most instances too incomplete and in some instances were only a portion of the data readily available.

Concrete Bridges: Relative Advantages of Different Types.

It is interesting and instructive to study the different types of concrete bridges which have been designed by engineers to suit various conditions. There is, of course, a tendency on the part of some designers to favor a certain type of bridge and to advocate its use without carefully analyzing the advantages possessed by other types. This tendency may easily lead to designs which are neither economical nor adequate under the existing conditions. To get the best results requires careful study and the consideration of several different types of bridges.

The flat slab type has recently been used in a number of cases. The most important examples of this type of bridge are the Bush and Gunpowder River bridges recently completed on the Maryland Division of the Pennsylvania Railroad and described in our issue of Feb. 11, 1914. These structures cross narrow inlets of the Chesapeake Bay and have a combined length of 7,01[illegible] ft. Numerous borings [illegible] no [illegible] foundation material was available, [illegible] it below the [illegible] of soft mud and sand and [illegible]. After comparing the first cost and the cost of maintenance of several types of bridge and carefully considering the difficulty of obtaining a satisfactory foundation for each type it was found that the flat slab type, with piers spaced 26 ft. on centers, offered the best solution of the problem. The slabs were built as simple spans, and a noteworthy feature of their design was the effective arrangement of the main reinforcing rods. These rods were bent in the form of parabalas so as best to resist the stresses developed in the slabs.

The girder type of bridge has been used to a very considerable extent, especially for highway construction in some of the middle western states. It has proved to be a strong and efficient type for comparatively short spans. Reinforced concrete truss spans have been used in a few cases, but this type of construction possesses more novelty than practicability.

In this issue we are describing and illustrating the principal design features of the superstructure for an interesting reinforced concrete viaduct now under construction. This structure comprises a series of spans varying in length from 60 to 80 ft., each span of which consists of two cantilever arms on reinforced concrete piers. Each pier and its two balanced cantilever arms constitute a separate unit as regards carrying dead and live loads. The superstructure consists of a reinforced concrete beam-and-slab floor supported by four curved reinforced concrete ribs. The curved ribs give the structure the appearance of a series of arches. The unstable character of the subsoil and the probability of unequal settlement of the piers led to the adoption of this type of construction.

The most common type of concrete bridge, and the one for which concrete seems particularly well adapted, is the arch. The fixed type of arch has generally been used, but there is an increasing tendency to use the three-hinged type in an attempt to overcome some of the difficulties encountered in the use of the fixed arch. For long spans and for arches where it is not possible to obtain a good rise the three-hinged arch possesses some advantages over the fixed type, but for short spans and for comparatively long spans with good ratios of rise to span it can not be used to advantage.

The discussions of several engineers on the relative advantages and disadvantages of the fixed and three-hinged types of arch are given in this issue. The relative efficiencies of ribbed and solid soffits and of divided and solid piers are also discussed. These comments were brought out by a paper, "Concrete Bridges: Some Important Features of their Design," an abstract of which appears in our Nov. 26, 1913 issue. In this paper the author asserts that the three-hinged arch possesses the following advantages over the fixed type: [illegible] a simpler analysis, [illegible] where rock or comparatively [illegible] foundation is at too great a depth [illegible], and (c) the temperature [illegible] eliminated. Advantages are [illegible] for the [illegible] soffit and for the [illegible]. Discussion is needed on [illegible] as there are still many [illegible] relating to the design and construction [illegible].

[illegible] that the three-hinged [illegible] advantageously used [illegible] and the rise is small, [illegible] largely eliminated [illegible] settlement of the piers [illegible] the hinges. The [illegible] the difficulty of se[illegible] construction details, however, are important items which may overbalance the direct saving of concrete. Steel, bronze and stone hinges have been used in the construction of existing arches. The friction developed in stone hinges may cause stresses in the arch which differ considerably from those computed. In the paper referred to it is stated that a ribbed soffit is usually more efficient than a solid one and that the I-section rib is to be preferred to the rectangular rib for spans above 200 ft. We doubt the advisability of using I-section ribs with sharp re-entrant angles, as the construction difficulties will usually overbalance the saving in the quantity of concrete required. It is almost impossible to design and construct a practical system of centering which will not deflect sufficiently to damage the flanges of the concrete ribs. The divided and braced types of pier may result in a considerable saving of material, but the intricate formwork required will act to reduce the actual saving in cost. Where diaphragms are used to connect the separate pier shafts they may serve to collect the drift at times of floods and thus greatly restrict the clear waterway. We sometimes hear designers complain of the calculations involved in the design of fixed concrete arches; but the experienced designer will find little difference in the amount of work required by the fixed and three-hinged types, and this item should not in any way affect the decision as to which type of arch should be adopted. More data are needed as to the relative efficiencies and costs of these two types of arch.

Explosions in Sewers.

Recent explosions in large sewers, particularly those which occurred at Pittsburgh and Detroit, have again drawn the attention of engineers to the causes of such explosions and to the preventive measures available. On account of the present interest in the subject we are giving, in this issue, several illustrations of the varying nature of the causes of sewer explosions and of the methods employed in various cities in an effort to prevent their occurrence. Underground wire conduits also are subject to explosions of similar nature and are so considered in the article to which we have referred.

The most spectacular as well as the most disastrous sewer explosion ever known in this country was that which occurred at Pittsburgh late in November of last year. Since this explosion serves well to illustrate the damage which may be sustained in accidents of this character, its more important features are here reviewed. The sewer affected is an 8-ft. trunk sewer draining an area of 1,700 acres and serving a population of about 50,000 people. Its dry weather flow is about 15,000,000 gals. per day. Varying degrees of damage were sustained by this sewer in a section 5,300 ft. long. A portion of the damaged length is 8x10 ft. in section and is made up of brick and stone; the balance is a circular, 3-ring brick sewer, 8 ft. in inside diameter. The sewer, in the affected section, was generally split at the crown of the arch, and the arch was flattened at most places and in some lengths demolished. The damage to the sewer section was confined almost wholly to the portion above the springing line, the invert being practically unharmed. The effect of the explosion extended for some distance up the laterals connecting with the trunk sewer. Much paving was torn up, manholes were demolished or badly shattered, many window lights in adjacent buildings were broken, and several buildings near the sewer were damaged by the explosion or the settlement which followed it. The physical problem of restoring the sewer to service was accompanied by the pressing necessity of protecting property from the effects of flooding, of maintaining street and railroad traffic, of keeping in operation affected industrial works, and of protecting the health of the people in the drainage area. The cost of repairing the sewer will exceed a quarter of a million dollars. The exact cause of the explosion has not been determined and, we are authoritatively informed, no official investigation to determine the cause will be made. It is significant to note, however, that the great majority of all automobile garages in the city are located in the drainage area of the sewer, as are also many dry-cleaning and paint manufacturing establishments. Acid waste from steel pickling vats discharge into the sewer, and a Pintsch plant and several large gasoline storage tanks are near the sewer line.

The Detroit sewer explosion, which occurred late in January of the present year and which will cost in the neighborhood of $7,000 for repairs, has been attributed to gasoline from automobile establishments. The affected sewer was drained at intervals along a section 19,000 ft. in length.

The subject is one of growing importance as the two instances just mentioned clearly indicate. Measures to prevent accidents to men engaged in cleaning sewers, and to prevent damage to sewers and adjacent property, resulting from explosions, will doubtless shortly assume more definite shape along the following lines: Specific city ordinances prohibiting the discharge into sewers of imflammable or explosive liquids or gases of any kind will be enacted or improved, and enforced; carefully conducted investigations will be periodically made to insure compliance with the provisions of such ordinances; efforts to secure better ventilation of sewers, particularly immediately before entering them for any purpose, will be made; and, finally, the striking of matches, or the carrying of any flaming light into sewers, will be prohibited. The article to which we refer clearly indicates the lines along which good practice in sewer maintenance, so far as the prevention of explosion is concerned, will crystallize.

General

The Fee System at Oakland, California, of Reimbursing the City for the Cost of the City Engineering Department.

The city of Oakland, Cal., supports its city engineering department by fees collected from other city departments and from private individuals. All employees are salaried and the salaries and other expenses, such as field and office equipment and supplies, are paid by the city. The city then charges for all work done in private or public service, the fees fixed by ordinance and these fees are credited as income against expenditures in salaries, etc. This system of accounting has been in force since July 1, 1911, and in the report of the Department of Streets for June 30, 1913, are given some data in the operation of the system which we summarize here:

ORGANIZATION AND SALARIES.

The Commissioner of Streets is in charge of the Department of Streets. The Superintendent of Streets is ex-officio City Engineer. The engineering department is under the Deputy City Engineer, and not only handles the engineering work for the Department of Streets, but also for the Harbor Department and all other departments of the city. The organization of the City Engineer's office is shown by chart, Fig. 1. This chart shows the whole scheme of fixed responsibility, which is the chief object desired in arranging the organization as shown. The results obtained have been very satisfactory. All orders are passed out through the proper heads which eliminates friction and confusion and gives a means of checking efficiency and of recognizing ability and good service. Vacancies in the technical service of the department are filled by Civil Service examinations and an eligible list of applicants is maintained for filling such vacancies as may occur from time to time. The organization and salaries paid as fixed by city ordinance are as follows:

1 Deputy City Engineer, at a salary of $3,000 per year.

4 Assistant Engineers, at a salary of not less than $1,980 and not more than $2,400 per year each.

3 Office Engineers at a salary of not less than $1,620 and not more than $1,920 per year each.

1 Field Engineer, at a salary of not less than $1,620 and not more than $1,920 per year.

1 Junior Engineer, at a salary of $1,680 per year.

4 Instrumentmen, at a salary of $1,500 per year each.

2 Instrumentmen, at a salary of $5 per day each.

14 Draughtsmen, at a salary of not less than $1,320 and not more than $1,500 per year each.

1 Assistant Draughtsman, at a salary of not less than $720 and not more than $840 per year.

10 Chainmen, at a salary of not less than $900 and not more than $1,080 per year each.

4 Chainmen, at a salary of $3 per day each.

1 Senior Bookkeeper, at a salary of not less than $1,200 and not more than $1,500 per year.

1 Junior Stenographer, at a salary of not less than $780 and not more than $1,080 per year.

1 Senior Clerk, at a salary of not less than $1,200 and not more than $1,500 per year.

In addition to the above, there is assigned to the engineering department a right of way agent, under salary by the city, whose duties are to obtain by voluntary dedication such miscellaneous rights of way and street openings as may be necessary for the public welfare and convenience. This arrangement has resulted in considerable saving to the city as compared with the method formerly employed.

FINANCIAL STATEMENT.

As indicating the financial workings of the system the income and expense account of the department for one year are given. The total cost of maintaining the department was as follows:

Salaries	$53,296.82
Office supplies	1,301.03
Office equipment	664.89
Field supplies	621.51
Field equipment	1,004.80
Total	$56,889.05

Under the head of office supplies are included such articles as drafting material, blue prints, pens, pencils, ink, pads and such other articles necessary for general office use. Under office equipment are included such permanent articles as drafting tables, filing cases, calculating machines, of which the department purchased two at a cost of about $550, planimeters, pantographs, etc. Under field supplies are included such articles as stakes, monument castings, car fare, etc.; and under field equipment are included such permanent articles as surveyors' transits (three having been purchased at a cost of $750), levels, rods, tapes, plumb-bobs, etc.

As a partial offset to the amount paid for maintenance, by the city, the following items are taken from the department books:

Fees collected on private lot surveys	$ 2,861.75	
Fees collected on private grade surveys	355.90	
Fees collected on street openings	233.82	
Fees collected by street department on street work covered by engineers' certificates	36,770.51	40,221.98
(Both public and private contracts.)		
Work performed for school department	2,183.19	
Work performed for park department	965.38	
Work performed for playground department	67.20	
Work performed for depart-		

ment of public health and safety—		
Fire department	7.50	
Police department	11.80	
High pressure fire system	245.18	
Work performed for civil service board	136.50	
Work performed for municipal water district	311 21	
Help assigned to the department of public works (harbor division)	4,079.87	8,008.13
Total		$48,230.11

No account is taken in the foregoing of the amount expended for the general operation of the office, of the cost of miscellaneous work ordered by the City Council or of the labor performed ($7,840.18) in bringing the maps and records of the office up to date.

FEES.

For those interested, the following is the fee ordinance under which this department operates:

Section 1. The fees to be charged by the City Engineer of the City of Oakland shall be as follows:

Subdivision "A"—For the survey of any lot, having four straight sides and being outside of the fire limits of said city, the fee to be charged shall be $12 for 50 ft., or less, of frontage and 5 cts, for each front foot additional. Survey to include diagram thereof and the setting of four stakes, additional stakes to be charged for at the rate of 50 cts. each.

Subdivision "B"—For the survey of any lot, as described in Subdivision "A," but lying within the fire limits of said city, the fee to be charged shall be 50 per cent greater than as fixed in said Subdivision "A."

Subdivision "C"—For the survey of any irregular-shaped lot (lot having more than four straight sides) the fee to be charged shall be at the rate of $20 per day for each field party of three men, and $8 per day for each office man; excepting, however, that the fees for irregular-shaped lots, as herein defined, shall, in no case, be less than the fees established upon a frontage basis.

Subdivision "D"—For giving official line and grade of curb or sidewalk in front of any lot of 100 ft., or less, frontage, $5 if done in conjunction with a lot survey and if done otherwise, 6 cents per linear foot of curb or sidewalk with a minimum charge of $8.

Subdivision "E"—Upon street work under the general street laws of the State of California, the cost of which is to be paid for by a frontage assessment, or upon street work, the cost of which is to be paid for in such manner as may be agreed upon between property owners and contractor (private contracts), the following shall be the fees, to-wit:

For setting stakes once upon:

Sewering or re-sewering, $6 for the first 100 ft., or less, with 6 cents for every additional foot of sewer.

Catch basins, $5 each.

Grading or re-grading, 7½ cts. per linear foot of street, measured along the center line.

Grading and curbing or re-grading and re-curbing, 10 cts. per linear foot of street, measured along the center line.

Curbing or re-curbing with wood, 3 cts. per linear foot of curbing.

Guttering or re-guttering, 3 cts. per linear foot of gutter.

Curbing or re-curbing with granite, cement or concrete, 5 cts. per linear foot of curbing.

Sidewalking or re-sidewalking, 6 cts. per linear foot of sidewalk.

Culverts, 6 cts. per linear foot of culvert.

Crosswalks, 6 cts. per linear foot of crosswalk.

Grading, curbing with wood and paving with macadam or oil macadam, 15 cts. per linear foot of street, measured along the center line.

Re-grading, re-curbing with wood and re-paving with macadam or oil-macadam, [illegible] cts. per linear foot of street, measured along the center line.

Paving or re-paving with macadam or oil-macadam, 5 cts. per linear foot of street, measured along the center line.

Paving or re-paving with macadam or oil-macadam and curbing or re-curbing with wood, 9 cts. per linear foot of street, measured along the center line.

Paving or re-paving with material other than macadam or oil-macadam, 5 cts. per linear foot of each line of stakes, measured lengthwise of the street.

Masonry work, 5 per cent on the cost of construction, as shown by the contract.

For plan, on private contracts, $5.

For assessment diagram and final certificate, $5.

No fee shall be charged on re-dressing the surface of improved roadways, unless stakes are set, in which case the fee shall be the same as herein provided for paving or repaving with macadam, or oil-macadam.

Resetting of stakes disturbed or destroyed by the contractor shall be at the expense of said contractor.

Subdivision "F"—Upon street work under the general street laws of the State of California, the cost of which is to be paid by a district assessment, the fee shall be 5 per cent of the cost of construction.

Section 1½. Fees for the following field or office work shall be as follows:

For such miscellaneous engineering, surveying or drafting work not specifically covered by Sections 1, 1½ or 2 of this ordinance, the fees to be charged shall be at the rate of $20 per day for each field party of three men, and $8 per day for each office man.

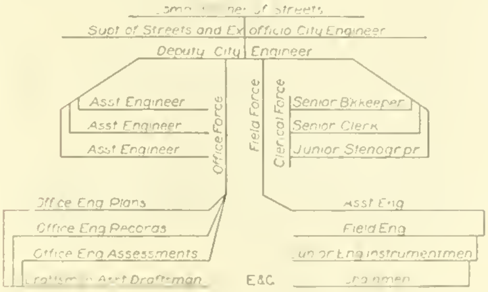


Fig. 1. Organization Chart, City Engineer's Office, Oakland, California.

For copies, additional to those required by Section 1 of this ordinance, of survey diagrams or street work certificates, $1 for each copy.

For blue print copies, additional to those required by Section 1 of this ordinance, of such tracings or negatives of maps, street improvement plans, assessment diagrams, etc., as are part of the office records of the City Engineer's office, 50 cts. for the first square foot of each of said copies and 5 cts. for each additional square foot thereof; no copy to cost less than 50 cts. and no blue prints to be made from other than office records.

For plans and specifications of work for which the City Council, or other authorized municipal body, is asking for bids, the City Engineer is hereby authorized to collect from prospective bidders on said work the sum of $5 for each such set of plans and specifications. For a period of 30 days after the time fixed for receiving bids on the work, said sum of $5 shall be considered merely as a deposit. In case said plans and specifications are not returned within said 30-day period, then they shall become the property of the holder thereof and the $5 held by the city shall be considered as payment therefor.

Section 2. Fees for street work under the general street laws of the State of California, on which the resolution of intention has already been passed, fees for street work on private contracts on which permission has already been granted and fees on lot surveys already ordered, shall be in accordance with the provisions of such fee ordinances then in effect.

First Subway in South America. The first section of a new subway, said to be the first in South America, has just been completed at Buenos Aires, and opened to public service. This subway is double tracked and equipped with cars of the most modern construction. The stations are distinguished by varying color schemes, as are those now in use in the Washington street subway in Boston.

Sieving Tests with Standard Cement Sieves.

Tests made by the Bureau of Standards on cement sieves submitted by two well known manufacturers lead to the following conclusions, in a report by Messrs. R. J. Wig and J. C. Pearson of the Bureau:

(1) Employing the present standard method of sieving, the greatest attainable accuracy in single fineness determinations of normal Portland cement on a standard No. 200 sieve, that is, the greatest attainable accuracy in checking uniformity of samples is about 0.2 per cent.

(2) "Standard" No. 200 sieves may differ in their sieving values by considerable amounts, such that their corrections to the ideal No. 200 sieve may be at least as great as 0.7 per cent.

(3) Errors of at least 0.5 per cent may be looked for in single fineness determinations of normal cements on a standard No. 200 sieve when made in the usual routine manner.

(4) Deviations exist in the sieving values of "standard" No. 100 sieves, of a magnitude, roughly, one-half the corresponding values for No. 200 sieves as given above.

(5) "Personal equation" appears to be appreciable in hand sieving, as in most laboratory operations, the observed values being as great as 0.3 per cent.

(6) The rating of a sieve by some system of demerits assigned from direct measurements appears to be an interesting possibility, and worthy of further study. Should a system be worked out to give reliable indications say within 0.2 per cent or 0.3 per cent of the observed sieving value of a sieve, it will add greatly to the value of the certificate now furnished with standard sieves.

It seems evident from the foregoing that both sieving tests and the interpretation of measurements on sieves are subject to considerable discrepancies, and the question arises as to whether some other more reliable method of determining fineness can not be made available. The sieve at best is a measure of the *coarseness* of finely ground material rather than the *fineness* and experiments now in progress at the Bureau of Standards indicate that air separation will offer a more satisfactory means of determining fineness than mechanical sieving.

In conclusion it may be stated that a tolerance of 1 per cent from the specification should be allowed with the No. 200 sieve and 0.5 per cent from the specification with the No. 100 sieve, every care being taken to conduct the test in strict accordance with standard methods. These tolerances should be considered as minimum values since they are based upon the results obtained by careful and experienced observers; therefore it should be emphasized that greater differences are possible in ordinary routine testing.

Plant and Drilling Methods Employed in Constructing a Mine Drainage Tunnel.

The tunnel described is 9x7 ft. in section and 7,030 ft. long; it was constructed for the Lehigh Valley Coal Co. by the Portland Contracting Co., Pittsburgh, Pa. The tunnel is wholly in rock, ranging from hard, close-grained, dark conglomerate, pitching at an angle of 15°, at the inside end, successively to coarse-grained conglomerate, blue stone of close grain, and small, red shale, at the surface end of the tunnel. The pitch of the rock flattens out from the inside end, until it is practically horizontal when the surface is reached.

The coal company awarded the contract on the bonus-penalty basis, in order to secure as rapid progress as possible. A certain bonus was offered per day, for each day the tunnel was finished ahead of one year's time and a like penalty per day was announced for each day over one year, required to complete the work.

EQUIPMENT AND ORGANIZATION.

While the work on the approaches to the tunnel was proceeding, the contractors established a power house and compressor plant, consisting of two Sullivan, cross-compound, belt-driven air compressors, with air cylinders 18 and 11 ins. in diameter, by 14-in. stroke, with a rating of 680 cu. ft. of free air per minute, at 165 R. P. M. These compressors were operated by belt from motors. The motors were of the 440-volt, alternating current type, of 150 H. P. each. On account of the hardness of the rock at the inside end, an air pressure of 120 pounds per square inch was maintained at the receiver throughout the entire 24 hours from the time the compressors were started until the work of tunnel driving was completed.

A 5-ft. exhaust fan was installed at each end of the tunnel, to provide the best possible ventilation. An 18-in. asphalted pressure ventilating pipe was carried as close as possible to the heading, so as to clear out the foul air resulting from the blasting as soon as possible.

A boarding house, providing good beds and good food, was established at each end of the tunnel.

The tunnel was driven three shifts per day, including Sundays and holidays. The labor force was made up as follows: 1 superintendent; 2 foreman; 6 chargemen; 18 drill runners; 18 helpers; 6 muck bosses; 24 muckers; 4 drivers; 3 dumpmen; 1 stableman; 2 compressor runners; 2 to 4 blacksmiths; 2 to 4 blacksmith helpers; 1 timekeeper, and 1 clerk.

TUNNELING METHODS.

The bonus-penalty work was started with the actual driving of the tunnel, on July 7, and included 6,800 ft. Work was carried on Sundays and holidays, and on the completion of the tunnel, on June 7, 1913, the contractor had finished 34½ days ahead of time, allowing for 454 hours of delay, for which the coal company was responsible, caused by high water, during January, March and April.

At the outside end, Sullivan 2⅝-in. differential-valve rock drills were employed, mounted on double-screw columns. These were replaced later, when the rock became harder, by 3⅝-in. machines. Two drills were ordinarily used in each heading, but when unusually hard rock was encountered, three drills were regularly employed. When the third drill was required, this work was done by the 2⅝-in. machine, to drill what would ordinarily have been dry holes. Drills of another make (also 3⅝-in.) were used on the inside end.

The sketches, Fig. 1, show the round which was employed. Sketch B, 21 holes, shows the hard rock round, and sketch No. A, 17 holes, shows the round in softer rock. The cut holes were drilled 8 ft. deep, and the relievers and side holes 7 ft. deep; a round thus ranging from 125 to about 160 ft. of holes.

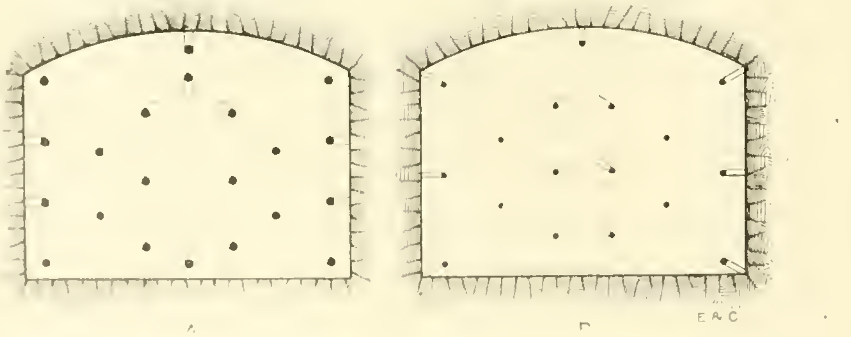


Fig. 1. Sketches Showing Drill Rounds Employed in Driving a Mine Drainage Tunnel.

The drilling crew consisted of three drill runners, three helpers, a muck boss and four muckers, making eleven men, to which should be added one chargeman, one driver and one dump man. The holes were fired by an electric battery, the cut holes being fired first, then the relievers, and then the rib holes. To pull the round 100 lbs. of 60 per cent dynamite were required, giving an average advance per shift for the entire work of 3.4 ft. per shift or 10.3 ft. per day of 24 hours for each heading. The average includes all losses of time. The actual rock pulled ran to 5½ ft. per working shift.

Slick sheets were placed in front of the face before each round was fired, to facilitate shoveling. The muck was removed by cars hauled by mules, each car being 8 ft. in length by 4 ft. wide and 4 ft. high. Owing to the solidity of the rock, and the fact that it was to be used for drainage only, no timbering was required, and no lining was placed.

The best month's progress for a single heading was 387 ft., driven by Sullivan 3⅝-in. drills in the south end in bluestone, in May, 1913, and the best progress for both headings was also in May, amounting to 754 lin. ft. of tunnel. The following table shows the progress by months, from April, 1912, to June, 1913, inclusive:

	North End. Feet.	South End. Feet.	Total. Feet.
April, 1912	00	80	80
May, 1912	115	94	209
June, 1912	186	23	209
July, 1912	228	000	228
August, 1912	219	186	405
September, 1912	252	192	444
October, 1912	294	315	609
November, 1912	321	372	693
December, 1912	309	309	618
January, 1913	327	342	669
February, 1913	309	351	660
March, 1913	351	300	651
April, 1913	327	240	567
May, 1913	367	387	754
June, 1913	...	...	234
Total			7,030

The data from which this article has been prepared are given by M. E. Reed in "Mine and Quarry" for January, 1914.

A Device for Recording at a Distance the Productive Operations of Machines.

(Contributed.)

A device which will record in the manager's office the operation of concrete mixers, steam shovels, hoists or other machines employed in construction is illustrated by the accompanying sketch. This device is called a "Productograph," and will record the work of as many machines as there are provided recording pins and counters. The sketch shows a five-pin machine; other standard sizes have ten pins and twenty pins. Each pin records the operation of one machine and the five or more machines whose operations are being recorded may be of the same kind, as for example five concrete mixers, or each may be a different kind, as for example, one concrete mixer, one derrick, one steam shovel, one pump, and one hoist. The several machines also may be the same distance or different distances from the office and may be at any distance, within reason, from it.

As indicated by the sketch the record is made on a chart carried on a revolving drum; this chart is renewed each day. The chart is divided by vertical lines into spaces, one space for each machine, and by horizontal (red ink) lines into time spaces—one hour, one-half hour, one-fourth hour, and five minutes. When a machine begins operation the pin recording that machine makes a horizontal mark recording each operation; each hundredth mark or multiple thereof is extended or made longer than the others, thus facilitating collation and computation. Stoppages of the machines are indicated by cessation of marks or by blank spaces. To illustrate the application of this recording process the operation of a batch concrete mixer will be considered.

The ordinary batch concrete mixer is a machine which by the hand movement of one or more levers will within an approximately prescribed time feed a batch of practically dry gravel and cement, in proper proportions, into a continuously revolving drum, the latter then receiving the water immediately upon this gravel and cement, through the actuation of a lever by the operator. The drum is revolved for a stated time to allow for thorough intermixing of water, gravel and cement to make a uniform concrete. All these operations are left entirely to the operator. The machine is practically automatic except for the hand lever movements which apply power to the different points of the mixer. The operator sets what in his judgment seems the proper length of time for mixing. Four consecutive operations, i. e., lifting raw materials to revolving drum, the water feed, the mixing, and the delivery, will produce a batch of concrete in say about 1½ minutes, each charge in an ordinary building mixer being only two wheelbarrows full. At this rate of 40 batches per hour, the yield for a full day's run of 10 hours should be about 400 concrete batches. This is based on the assumption that the mixer has run the day through without interruption of any kind and has been fed steadily, and conscientiously looked after by its attendant. Now let us say that by actual count of the yield at the end of the day it is found that only 300 batches have been mixed, a shortage of 100 batches, equivalent to a loss of 25 per cent in the capacity of the mixer, or 2½ hours of running time. Under ordinary conditions and management one of two things will follow, either the day's record will be accepted without further inquiry as the best possible, or the responsible head will want to know the cause for the shortage. It may have arisen through laxity of operators in keeping the mixer fed regularly, or through defects in the mixer which caused a stoppage every now and then. In either case it becomes a matter of veracious report from the workman, and it is not likely that they will lay blame on themselves. Thus the real causes of shortage are vague and indefinite, and may remain uncorrected for days or weeks. If the shortage should be less than we have figured in this case, it would doubtless be passed by as too inconsequential, but suppose the condition should exist where there are ten or more mixers at work, each losing 10 per cent

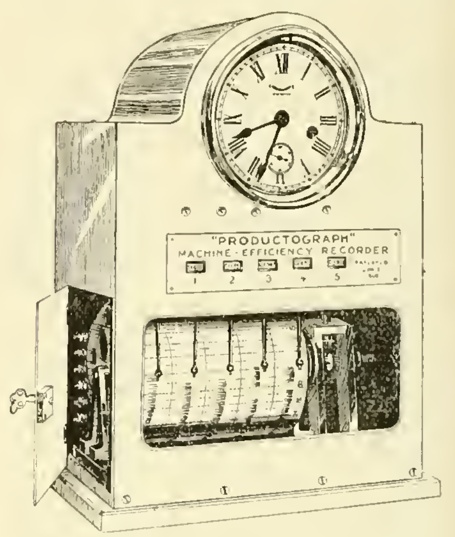


Fig. 1. Productograph for Recording the Operation of Machines.

of its capacity every working day? Then the seemingly immaterial item grows into a mountain of loss.

Now, against this uncertainty and looseness of procedure, let us suppose that by some device the mixer could self-record its every movement and stoppage, and its production at the very moment it yields it, batch after batch, making a permanent record of all these things in the office of the owner or manager, away from the observation of the workers. Such a record, comprehensively made, would reveal in the above case, for example, that the mixer stopped for a full hour at some time of the day. The office can take immediate cognizance of it at the very time it happens, and promptly learn the cause; it may be due to break off a piece of mechanism, necessitating repair. Right here comes the additional advantage in the ease with which an astute manager can discover if too much time has been consumed in the work of repair; at any rate he is sure to discover the breakage much sooner than it would ordinarily be reported to him.

Or, perhaps, the record will show frequent and irregular stoppages of five, ten or more minutes each. What more natural than to conclude that the workers are "soldiering," or that the operation of the mixer is so imperfect that it blocks or jams up now and then, and needs cleaning or removal of imperfectly mixed product. It must be evident that such a record, carefully analyzed and seriously followed up, will reveal the true facts, place the blame where it really lies, and make correction of the fault—whether in man or machine—a most simple matter. The record, made by hands not human, is dependable and strictly impartial. In the case of concrete mixers, "soldiering" should be especially discouraged because inaction of the chief operator means idleness of a number of other workmen who depend upon the proper operation of the mixer, possibly four or five men to every such machine.

In similar ways the performance of any machine used in construction work can be timed and recorded.

The "Productograph" is made by Slocum, Avram & Slocum, Industrial Engineers, New York City.

Cost of Drainage Drill Borings in Prospecting for Ore in Alaska.

The Alaska Treadwell Gold Mining Co., at Treadwell, on Douglas Island, Alaska, has been using a diamond drill the past year to prove up the deposits of gold ore on some of its properties. The drill employed is a Sullivan "Champion" screw-feed machine, with differential feed-gears, having a capacity of 1,500 ft. in depth, and removing a core 1⅛ ins. in diameter, employing standard A fittings; that is, hollow rods, core barrel, core shell, split spring ring lifter and bits.

Diamond drilling work began on April 6, 1913, at the "Mexican" mine, and continued until 4,318.5 ft. of drilling had been completed. All drilling performed was cross-cutting from the main drift of the mine. The space required for operation was 6x6x15 ft., the rods and core barrel being 10 ft. in length. The rock in which the drilling has been done is diorite, quartz, greenstone and slate. The diorite and quartz are very hard, and the greenstone has been ordinarily the best material for drilling. While not as hard as the diorite and quartz, it is still hard enough to provide very good cores. The slate encountered was very soft, but through it ran quartz seams, making very rough cutting, hard upon the machine and hard upon the diamonds. The diorite was so hard as to put a glass polish on a bit set with diamond chips, in from 4 to 6 ft. of drilling. The diamond setter, Mr. J. M. Gibeau, got good results by setting some small diamond chips, which had been saved by the mine from work done about 16 years ago; but although these were set up very sharp, the diorite rapidly polished them, so that progress in this material was very slow.

The following table shows the cost of five months, to Aug. 31. The distance drilled was 3,048.5 ft:

	Total cost.	Cost per ft.
Labor	$2,434.36	$0.798
Diamonds	1,941.62	.637
Repairs	46.61	.012
Supplies	253.66	.083
Assaying	8.26	.002
Power (estimate), two machine drills at $65.00 each	655.00	.211
	$5,339.51	$1.743

Number of karats used, 21.57.
Feet drilled per karat, 111.3 ft.
Carbon cost based on a value of $90.00 per karat.

No exact data are available on the cost of drilling for September, but the following is given as a close estimate:

	Drilling for the month, 584 ft.	Cost per foot.
Carbon loss	$181.22	$0.31
Labor	634.00	1.085
Power	131.00	.224
	$946.22	$1.619

During August the total amount of drilling was 900.5 ft. The carbon loss for August was $.125 per foot drilled. The total cost for that month was less than $1 per foot. These data are taken from an article by A. Schoenberg in "Mines and Quarry" for January, 1914.

Water Works

Influence of State Regulation Upon Economy and Efficiency in the Operation of Water Works Properties.

Contributed by Wynkoop Kiersted, Consulting Hydraulic and Sanitary Engineer, Kansas City, Mo.

The economical and efficient operation of a public utility, like a water works property, depends somewhat upon the conditions under which it is operated. In the past many public utilities of this kind were constructed under provisions of a limited franchise by promoters desiring to profit rather in the financing and construction of a plant than in the operation of the property. Little attention was given to planning its structural details. Usually the promoters and the city officials reached an agreement as to the source of water supply, the sizes of pipes in the streets, the number and distribution of hydrants and valves, the size and character of pumping equipment, reservoirs, standpipes and other details of the physical property, and finally as to the amount of hydrant rental and the schedule of rates expected to prevail during the life of the franchise. Generally the annual hydrant rentals in the aggregate approximated 6 per cent interest on the estimated cost of financing and constructing the plant. The rates for domestic consumers—usually those of neighboring towns—were accepted without regard to their application to the town in question.

If the capital obtained by an issue of bonds secured by deed of trust on the property and franchises, with hydrant rentals pledged for interest, was not sufficient to meet the cost of construction, additional money was taken from the sale of such portion of the [illegible] did not go as a bonus to the promoters or to the purchaser of the bonds. Usually the promoter's connection ceased soon after the completion of the original construction and the management of the property passed to the owners of the equity. As a rule during the early years of operation [illegible] economy prevailed. The earnings of the earlier years were often scarcely enough to meet the interest and operating expenses of the property, to say nothing of earning a return on the equity or of earning a replacement fund to take care of the property. This was particularly true in small and slow-growing cities, while in a rapid-growing city the demands for extensions following in rapid succession often embarrassed the management depending upon borrowed money with which to make the extensions.

In towns of slow growth this situation proved almost a struggle for existence, and frequently resulted in financial embarrassment to the owners of the equity and often serious loss to the owner of the securities against the property. But to the management in cities of rapid growth the embarrassment was temporary only, for as soon as the growth of the city proved substantial the prospects brightened sufficiently to attract outside capital, which enabled the management to finance the extensions to the property with but comparatively short delay, although unable at first to keep pace with the demands. Even this difficulty disappeared as the city increased in wealth and prosperity to an extent which permitted the management to anticipate demands and to build to a limited extent for the future.

The premature abandonment of operating units of plant or the early duplication of mains and pipe lines resulting from rapid growth did not necessarily indicate poor business judgment or poor engineering advice in the outstart, for the financing of the original project admitted of anticipating future requirements only to a limited extent. An attempt, then, to anticipate rapid growth would doubtless have resulted in overcapitalization and overconstruction to an extent which would have seriously embarrassed the original operating company. Every water works property has passed through a period of structural evolution when structural details were gradually modified and improved, high-grade machinery superseded low-grade machinery, mains and pipe lines were duplicated between the source of water supply and the distribution pipes of the city, large pipe headers of permanent construction were substituted for the original and irregularly laid network of pipes about the pumping station connecting the pumping machinery with the mains leading to the distributing system, improved methods of handling fuel were installed and various labor-saving devices took the place of those originally provided. The operating costs of such properties, passing through a similar process of evolution, gradually decreased per unit of water output as the mechanical details of the plant were improved and perfected, and as the methods of conducting the business of the company became better organized and systematized, until finally a point was reached where further change in structural design and methods of operation accompanied change in the mechanical arts as applied to the handling and the purification of the water supply, and the necessity for reinforcing parts of the property and for extensions in conservative anticipation of future growth.

In the past there have been several handicaps to the development of water works property along lines of economy and efficiency. Instances are numerous where the relations between the citizens and the management of the property became seriously strained by reason of poor water service. Doubtless action of the city council growing out of this strained relation may have carried the popular agitation beyond reasonable bounds and afforded an opportunity to withhold hydrant rentals from the operating company, the usual weapon of offence and defence of the municipality. Usually a controversy of this kind ended in compromise and reduction of hydrant rentals and a crippling of the water company financially.

The second handicap was occasioned by failure, deliberate or otherwise, of the water company to improve the plant to meet the necessary requirements of good water service, the management particularly a foreign management, sometimes preferring to spend money in contesting the claims of the municipality to an improved water service, with the hope of sustaining in court certain unwise technical provisions of the franchise rather than to expend money in improvements under good engineering advice. Such action, engendering intense animosity on the part of the citizens, usually resulted in litigation with heavy and fruitless expense to both sides.

The third handicap, the limited franchise, has at times prevented in one way or another a progressive development of the property to meet the increasing demands of a growing city. Both sides have used the limited franchise as a club with which to force concessions, particularly during the closing years of the franchise period, the one seeking extensions and improvements perhaps in excess of reasonable requirements, and the other refusing all improvements unless the city would agree to an extension of franchise.

Combining the financial embarrassment experienced during the financially lean years of the enterprise while acquiring business, with the neglect on the part of the business management of proper observance of engineering considerations, and with poor service arising from procrastination, enforced delay and political complications, together with the inability or unwillingness of the management to extend the property during the latter years of the franchise period, there was evidently no substantial foundation and little incentive for a truly economical and efficient management. This situation has been particularly emphasized by the fact that investment in a waterworks property, like that in many other public utilities, was considered altogether a private venture undertaken for private gain, under a specific contract with a municipality which, by reason of financial inability or otherwise, could not or would not provide its own water supply. Whenever the improvement of a deficient and impaired water service interfered materially with the fair return on the investment it naturally afforded a pretext for delaying such improvements, particularly if certain unwise or conflicting provisions of the contract offered an excuse for delay on legal or purely technical grounds.

Fortunately all managements were not so short-sighted. In any event the inevitable litigation following such a construing of the terms of a franchise in the face of poorly rendered water service almost inevitably resulted in withholding hydrant rentals and frequently in permanent reduction of rentals, regardless of whether the reduction was justified or not from the standpoint of fair return. The incentive on the one hand to evade responsibilities and on the other hand to inflict a penalty for attempted evasion was a natural result of such a situation. The outcome of such controversies might have been quite different in many instances could the courts have compelled the presentation of evidence to establish a true and comprehensive state of facts, and finally, when warranted by the facts, so construed and modified the contract as to compel the company to expend the money required to properly improve the property and the city to pay a fair return upon the investment thus made.

Doubtless such conditions as these described have contributed to the sentiment favoring the creation of the state public utility commissions, clothed with both judicial and administrative powers in the regulation of the public utility corporations of the municipality and the state. Such a regulation as this may be considered as trespassing upon private rights and upon the home-rule theory of government as applied to the administration of municipal affairs. Nevertheless it is an improvement over old-time methods, and until the management of municipal affairs shall have improved in some particulars it should be considered a welcome change.

A public utility law properly drawn and administered would eliminate the necessity of the limited franchise. A commission organized under such a law, clothed with authority to regulate rates and to fix values for rate making, or for purchase whenever the city desires to undertake municipal management, and to regulate the rates of properties under municipal management, is in a position to protect the citizens of any community against unfair or greedy private management, and to insure a fair return upon water works investments, and when free from political incentives itself to check the political mismanagement of municipal properties. Such a commission should be and usually is much better constituted than many of our courts to consider the business and technical details of construction and management of public utilities, because of the opportunity to embrace in the personnel of the commission the business and technical as well as legal knowledge required in sifting evidence and weighing the questions relating to the development and management of public utilities, and the power to call to their assistance skilled technical knowledge as occasion demands.

The regulation of public utilities in this manner really creates a sounder basis for the economical and efficient management of a water works property than that which has prevailed heretofore. It places the public utility company in the relation or capacity of trustee of a property expected to be owned, controlled and operated in the interest of the public, and it more readily insures fair and legitimate returns upon the capital invested, and compensation for management under the trusteeship until the municipality purchases the property at its fair value. This relation of trusteeship on the part of the owners of a water works is emphasized from the fact that a water works property is essentially one which should be owned, controlled and operated by the municipality, first because the character of the water supplied to the citizens is closely related to the health, comfort and prosperity of the community, and second because of the heavy expense attending the construction of a public utility of this kind. This expense, although naturally heavy, is gradually increasing by reason of the increasing demands for water purification and the high standards of purity required from the operation of such plants.

Regulations of the kind described may from certain points of view appear to decrease the hazards to which investment in water works properties have been heretofore exposed by reason of the fact that it eliminates the source of many troubles and contentions, and provides for arbitration by a distinterested commission of all grievances and contentions on the part of either the public utility corporation or the city. However, from another point of view the hazards may not appear to decrease, for the power which created the public utility commission and subjected public service corporations to regulation can also as readily dispense with the commission and restore the old relations of public utility corporations with the state and the municipality, with all the attending delays, uncertainties and complications in court procedure. So long as the state utility commissions use their powers fairly, impartially and with discretion towards the public, and do not invade too far the home-rule principles of government, this backward step is not likely to be taken. Nevertheless so long as such uncertainty exists, it entitles the investor in a water works property under private management to a larger rate of return upon his investment than upon an investment in property like real estate, which is exempt from such uncertainty.

A rate of return on an investment secured only by the physical property which that investment represents, should be more than the interest upon municipal, county and state securities. Moreover, adequate compensation must be allowed the management for acting as trustee or custodian of the property until the city assumes the management and control. Under these circumstances the management is obliged to make investments in plant judiciously and conservatively in order that the provision for the extra return upon capital investment and compensation for services shall require rates both for the public fire service and for private consumption which are not burdensome to either the property holders or the community. Accordingly it cannot be expected that the investments in plant and the changes of plant to meet changes in the mechanical arts will anticipate future requirements and will embrace as substantial structures, and will be accomplished with as much promptness as when under well-organized and efficient municipal management.

The obligation is threefold, namely: upon the public utility management to give good service and to expect no more return and no more compensation and profit than is consistent with faithful discharge of a trusteeship; upon the public to provide the fair return, compensation and profit; and upon the utility commission to see that both principals properly discharge their obligations.

The first step in the discharge of these obligations is the agreement upon a value of the public utility property and the fixing of rates which will be fair to the rate payer and the owner of the property and commensurate with the service rendered the public in each particular locality and under each set of physical and financial conditions. In the exercise of this duty in a broad and equitable manner the public utility commission will have to consider various aspects of the questions relating to valuation and rate making, among which are the following:

ASPECTS OF WATER WORKS VALUATION AND RATE MAKING.

Valuation.—(a) Original cost as a good index of structural value of a comparatively new and well-designed and constructed property.

(b) Cost of reproduction new of an existing plant based upon average prices of labor and material as necessarily the basis of computing structural costs in the absence of proper original cost data, particularly in connection with old property wherein replacement of units has been made progressively. Estimates upon this basis usually include to a greater or less extent certain unearned increments like that embraced in the enhanced value of real estate as compared with its value at the time of purchase, the enhanced cost of construction under present conditions as compared with those of the past, and such other enhanced values as result primarily from association with urban conditions and urban development.

(c) Value based upon reproduction new, embracing structural costs, accrued value due to urban conditions, certain overhead charges to cover items of cost indeterminate with precision, the cost of reproducing the business of the property, rights of way, and sometimes water rights in sections of the country where the scarcity of water renders such rights peculiarly valuable; but not including for rate-making purposes franchise value.

(d) Overhead charges, intended to include the costs of preliminary engineering and legal investigation, costs of organization and similar necessary preliminary expenses; interest during the period of construction; engineering and supervision of construction; contingent expenses due to accident or unforeseen contingencies; insurance, taxes and administration; delays affecting organization and interfering with the general progress of the work. Overhead charges when not included in unit costs are usually estimated as a percentage of structural costs. The amount of this percentage is necessarily larger in connection with estimates of large properties in large cities involving many obstacles, interruptions and uncertainties of various kinds than it is in connection with properties in small cities and towns where there are likely to be but few delays and few or no serious obstacles or uncertainties. Usually overhead charges are underestimated in connection with the engineering, administration and contingency cost of large properties. Insofar as overhead charges embrace interest on expenditures during construction a commission may consider such interest allowance as absorbed by the unearned increment of value of items like real estate, water rights, pavement over pipe originally laid in earth, and the like, particularly in a rate-making valuation.

(e) Business value as distinct from structural value. It is difficult to discriminate between business value and franchise value, and even more difficult to definitely estimate the element of value termed business value. The past losses during financially poor years evidently cannot be a measure of such value as it would inevitably make the capital value of a property operated under abnormally low rates greater than that of the same property operated under high rates. Moreover if it is right to charge past losses to capital account it is equally right to charge past surplus earnings against capital account. The situation would then become anomalous—namely, if the rates were too low the charge to capital account would be large; if the rates represent a fair average, then losses and surplus earnings would offset, and there would be little or no charge to or against capital account; and if the rates were so high as to produce a surplus continuously a heavy deduction from capital account would result. It would seem, therefore, that there is no logical connection between past losses or past surplus earnings and business value. Either both loss and surplus must be considered together and on opposite sides of an account, or both omitted altogether. It would seem that a logical way of computing business value is to allow in the capital account a sufficient amount to cover interest and maintenance upon investment in physical property for a sufficient period of time after the completion of construction as will admit of the necessary business organization and the acquirement of the patronage existing at the time of valuation. Such aid as can be brought to the assistance of the judgment by an analysis of existing earnings in connection with cost of reproduction new should be proper in arriving at the business value.

(f) Such elements of value as water rights, which arise for consideration in particular cases, must be considered in connection with local conditions under the interpretation afforded by relevant court decisions in the locality in question.

Deductions.—Shall deductions be made for structural value in a rate case? A property wherein repairs of the operating parts and replacements of incapacitated units have been made properly as needed and the property otherwise kept in an efficient condition possesses a value for rate making but little, if any, below 100 per cent. On the other hand, a property which is neglected and in which no more repairs have been made than to barely keep it in operation, and in which few if any replacements have been made to maintain its capacity and efficiency, is a depreciated property, and in the event of purchase and sale the value as estimated would be depreciated by an amount at least necessary to restore that property to an efficient operating condition. But should the value of a property be similarly depreciated for rate making? An investor who keeps his property in a thorough state of repair, makes the replacements as needed out of earnings, and looks after and operates the property as though it were perpetual, and then earns no more than a fair return upon his investment should earn continuously on a 100 per cent structural value. Rates based on less than a 100 per cent value would either cause loss to the owner or afford an incentive to neglect necessary repairs and replacements in order to avoid loss. The owner of a neglected property in order to continue in business must restore the property to an efficient operating condition by making the necessary repairs and replacements. These restoration expenditures actually add nothing to the capital investment in physical property. Were the rates to be based upon a value depreciated by the amount of restorative expenditures, [illegible] accrued depreciation in computing a purchase value, the return becomes inadequate and the incentive may be increased to neglect the property in the future even more than in the past. The conditions, although not parallel, are similar in both instances to the extent that the rates should be based upon a value admitting of efficient operation and maintenance, which for water works is [illegible] from a 100 per cent value. The well-managed property has a business value upon which the owner should also earn a return, while the neglected property can have little or no business value until acquired through good management. A public utility commission should have the power to compel high-grade maintenance and operation, and the restoration of a neglected property, and to penalize a management for profiting or attempting to profit through neglect of the property or the service, and at the same time to hold forth an incentive for good management. A municipality possessing the right to purchase the property at any time can terminate protracted neglect and inefficient management by purchasing the property through condemnation proceedings.

Rate of Return.—(a) Computations relating to the fair return should evidently discriminate between depreciable and non-depreciable property. Should not the non-depreciable property be further classified as property possessing value only as a part of the plant, and property possessing value independent of the plant on account of its relation and connection with city progress and improvement?

(b) The rate of return upon the depreciable property should be at least an amount equal to that on safe investments in the locality plus an amount to cover any additional hazards to which investment in the particular class of property may be subject, plus an amount to cover general maintenance.

(c) The rate of return upon non-depreciable property, of value only as a part of the public utility, should evidently be the same as in the preceding case, exclusive of general maintenance. Should the rate of return upon the other class of non-depreciable property, possessing value independent of the utility, be any more than that corresponding to the rate of return on secure and substantial investments in general city property?

(d) To what extent should the foregoing rates of return be increased to include a reasonable profit to the owners of a property under good management?

(e) By the term general maintenance is meant ordinary repairs and replacements of and substitutions for units of the property from time to time as may be needed to maintain the property in an efficient operating condition. Evidently no return to provide a sinking fund to repay the owner the original investment is required, because the replacement earnings expended in replacements and betterments perpetuate the property and maintain it practically at a 100 per cent value. But to the above rate there is an addition open for determination to cover expense arising from accidents, unforeseen contingencies and such similar expense as cannot be anticipated.

(f) The class of non-depreciable property earning the least rate of return embraces real estate, water rights and such other items as possess value entirely independent of the public utility with which they may be connected by virtue of association with urban development or because of a general demand and applicability to other useful purpose than the particular one to which they may be devoted at the time, and to the ownership of which no unusual hazard attaches.

ESSENTIALS OF EFFICIENT AND ECONOMICAL MANAGEMENT OF WATER WORKS PROPERTIES.

Summarizing what has been said, some of the essentials of a more efficient and economical management of waterworks property may be mentioned the following:

1. An unlimited franchise.

2. Unbiased and disinterested regulation in a manner which will insure to the citizens of a municipality good water service and such protection to the investor that the hazards surrounding the investment are sufficiently eliminated to invite a low rate of interest.

3. Sufficient earning capacity to cover operating expenses, taxes and repairs, to insure replacements and substitutions of units of plant as needed in order to perpetuate the property and to maintain as nearly as practicable a 100 per cent value of the investment indefinitely, to provide a margin for profit inviting to a good business management but discouraging to a wasteful management, and on the whole to compensate fairly for furnishing good water and good service, and for reasonable anticipation of future requirements in the extension of the property.

4. The determination of an earning value of the property which shall be fair and equitable to both the rate payer and the investor, and still discourage a neglectful management, and such as will admit of the investment in plant requisites for good service and good water.

5. The exercise of such administration over the investment of either the municipality or private investor in public utility property as will insure the expenditure of money in proper kind of plant and equipment, but will not interfere unduly with the questions of design of the property so long as guided by good technical advice.

6. The insistence that the waterworks management, whether under municipal or private direction, shall standardize, simplify and systematize methods of operation, and render uniformly a good water service.

7. The assurance of freedom from municipal competition on the one hand, and on the other hand the right of the municipality to purchase at any time upon due notice, at a value fixed by unbiased and disinterested appraisers, or to condemn the property in accordance with some defined legal procedure.

Use of Well Points for Unwatering Quicksand in Excavation, at Battle Creek, Mich., for Pumping Station, Suction-Storage Basin, and Clay Pipe Supply Line in Deep Cut.

In the construction of a pumping station, for an artesian well water supply at Battle Creek, Mich., a basin underneath the station, of 50,000 gals. capacity, and in laying a clay pipe gravity supply line to the basin in trenches ranging in depth from 12 to 18 ft., a very difficult piece of quicksand excavation was encountered. The methods employed in handling the excavation on this job were described by Mr. W. W. Brigden, Superintendent and Engineer of the Department of Public Works, in a paper before the latest annual meeting of the Michigan Engineering Society. The paper, as here given in substance, contains valuable practical information on methods which Mr. Brigden has successfully employed in excavating trenches in quicksand and in laying clay tile pipe lines in such trenches, with special reference to the job above mentioned.

In overcoming the difficulties brought about by quicksand excavation more than ordinary care is exercised in sheeting the trenches, U-shaped guards to surround the pipe while being laid are used, and cradles of wood, gravel, piles, oakum, cloth rolls and other special contrivances for keeping the cement in the joints until well set are also employed. Cloth rolls, etc., when used, should go all the way around the joint, not the lower half only, since otherwise the ground water will rise and wash the cement out of the joints before it is set.

The main departure from ordinary practice in the work undertaken by Mr. Brigden was the driving of small wells 2 ins. in diameter along one side of the trench and connecting these well points by means of pipes 2 to 4 ins. in diameter. These latter pipes led to a steam pump or pumps, and pumping was continuous day and night, with traction engines for power. Wells on both sides of the trench would probably be better in most cases. A small diaphragm pumping engine on wheels, using gasoline fuel, was found to be of great assistance, especially where working room was limited. The pump was placed on the surface and the 3-in. suction hose could be used at any point below, since it was easily moved. Siphons operated by city water pressure through 2-in. pipes were also found very satisfactory. These pipes were placed a few feet above the bottom of the trench, close to the pipe layer, and discharged at the same level to a centrifugal pump which lifted the water to the surface.

The trenches excavated were from 3 to 4 ft. wide and from 12 to 18 ft. deep. The pipe laid was 12 ins. and 15 ins. in diameter Ordinary 2-in. x 10-in. yellow pipe sheeting 14 ft. long was used, the ledgers or rangers being 3-in. x 8-in., 14 ft. long. In cases where the trench was too deep for this length of sheeting, or the quicksand was more than ordinarily difficult to handle, Wakefield sheet piling made of three pieces of 1-in. lumber was also used. It was necessary most of the time to mow large amounts of swamp grass to stop up cracks between the planks, and at times cloth cement bags filled with sand were also used for this purpose.

Quicksand was found from 1 to 3 ft. below the surface and in most cases nearly to the bottom of the trench. There were innumerable boulders where the bottom of the trench came to or near the Marshall sandstone rock, which was the case the major portion of the time. In some cases these boulders were so plentiful as to form a continuous line for 30 or 40 ft., with only an inch or two between the stones, and again they were so large as to reach entirely across the trench and jut out on both sides to a distance of a foot or more. No way was found to care for the largest boulders except to drill holes in the top and blast them with dynamite. This process was objectionable as it almost invariably nearly wrecked the adjacent sheeting. When the boulders were small enough to be taken up by a derrick they were removed by that means.

When the excavation reached the sandstone rock conditions for pipe laying were generally much better than where the pipe was laid in quicksand, as in the latter case the quicksand often came in at the sides and also up through the bottom of the trench, displacing a pipe when it was almost laid. The conditions were also comparatively good when the pipe was laid in the glacial till which overlays the sandstone rock. This till contained most of the boulders. During the process of excavating one of the trenches where the sandstone rock was not reached, no pipe was laid for three or four days although constant attempts were made. It was while working on this trench that the ordinary system of trenching and pipe-laying was abandoned and 2-in. wells were put down 10 to 20 ft. apart, and reaching below the level of the trench bottom, to pump the water out of the quicksand. At times this was so successful as to enable the workmen to get within a foot or two of the bottom of the trench without any serious trouble from water, and in one instance, where the material was a very small-grained quicksand, the clay pipe was laid in the dry trench for about 10 ft. In all other instances, although the 2-in. wells were driven below the bottom of the trench, there still remained some free water which brought in quicksand so as to interfere in a measure with the pipe laying, but most of it was removed by the pumps, and without their aid some of the pipe could not have been laid.

While the driving of wells to remove water in quicksand is not entirely new, and in fact has been used a number of times, it is still a rare manner of doing such work, and, so far as known the method has not heretofore been undertaken in southern Michigan for the purpose of laying clay pipe. While clay pipe was used in these trenches it was for the purpose of carrying water from 5, 6 and 8-in. artesian wells to the basin under the pumping station, but the methods apply equally well to sewers, the only difference being that the pipe here under consideration was laid on a level grade. Water is pumped from the basin into the force main This method of getting water to the basin was enormously expensive, as much of the work cost $15 per lineal foot of trench, but it has many advantages, especially with the centrifugal type of pump that is to be used in the pumping station. All the sand, pieces of rock, and other material coming from the wells, pipes and manholes is deposited in the basin and is removed at convenient times by hand, so as not to abraid the working parts of the pumps. There is also opportunity for aerating the water for iron removal by means of weirs across the waterway in the basin, a little space for storage is also provided by the basin, and suction is not so easily lost as in direct pumping from wells.

With the experience gained in putting down about 1,000 ft. of trench it was discovered that several errors had been made. The well contractor supposed it was only necessary, where the bottom was in quicksand, to drive the well points a foot or so below the bottom of the trench and he drove the first three or four wells in that way. When Mr. Brigden arrived on the ground he objected to this mode of procedure and insisted that the wells be driven until the whole point, about 3½ ft. long, was below the bottom of the trench, and the contractor then drove them deeper. It was discovered later that even this was not deep enough and that a well point should probably be driven so its lower end would be 4 or 5 ft. below the trench.

The line of wells was laid out on one side of the trench so as not to interfere with the digging, which was done on the other side, and the wells were placed 4 or 5 ft. outside the sheeting. This was found to be too close to the trench for when the ditch was dug nearly to its bottom the quicksand between the well point and the bottom of the trench became the line of least resistance and air passed through from the trench to the point, thus greatly lessening the amount of water that could be removed, and, on a few occasions, letting in so much air that the water could not be raised the necessary 15 ft. until the pumps were placed in the very best condition or some of the poorer wells disconnected from the system.

A few of the well points or screens were stripped by striking boulders. Some of these were removed at the time of driving, but in the case of others the injury was not discovered and these wells of course were in poor condition to screen very fine sand and considerable quantities passed through with the water. The sand at times filled the pipes from the wells to such an extent as to stop pumping.

The writer supposed, when adopting the 2-in. well system, that it would be necessary to purchase some unusually fine screens for wells where not driven into the sandstone rock, but this was found unnecessary. The finest quicksand encountered in the trenching could probably have been handled well with the ordinary No. 60 gage screen, although No. 80 was used for that part of the work. It was necessary to send to Chicago for the No. 80 screens, as they could not be obtained in Battle Creek. The wells in those trenches where the bottom was in or close to the sandstone rock were driven without screens and to a depth that would shut out the quicksand, this generally being from 4 to 7 ft. into the rock.

Although not more than from 100 to 300 ft. of trench was opened at any one time the volume of water coming into the trenches, leaking through joints and into manholes and the basin was so great as to require constant pumping during the latter part of the construction, amounting to from 500,000 to nearly 1,000,000 gals. of water per day. Even when a 6-in. centrifugal pump was attached to one of the 8-in. wells and pumped to its full capacity, 2,000,000 gals. per day, the ground water in the immediate vicinity was not lowered more than 5 or 6 ft. and the difficulties of running the pump became so great, owing to throwing off the belt, losing suction, priming, etc., that the process was abandoned and the 8-in. well pipe was then carried up high enough to prevent any overflow into the trench.

The method of handling the 6-in. and 8-in. wells during construction is of interest. These wells flowed continuously near the level of the ground and, as valves were required on each one and the lines of clay pipe to the basin were 12 to 18 ft. deep, it was necessary to cut off the well pipes at that depth and to build a manhole at each well to make the valves accessible for repairs, renewals, etc. The flow from a well at the depths stated was at a rate of 600 to 1,000 gals. per minute and if it ran even for a few minutes would have ruined the trench and piping in the vicinity. At first there appeared to be no practical way out of the difficulty, but later it was solved by the use of a plumber's test plug. The plug was taken apart and a long handle or stem put in place of the short one ordinarily used. By this means two men could shove the plug down the pipe, turn the nut that expands the rubber disc and when it filled the well pipe the men could unscrew a section and screw on the valve. In most cases it was necessary to cut off the pipe with a hack saw; then by means of a short piece of pipe and a sleeve put on by means of a lead joint the valve was placed near the bottom of the manhole. By screwing 10 or 15 ft. of pipe into the valve, reaching the surface, the plug could be withdrawn and the valve closed after which the pipe above the valve was removed.

The rising of the ground water after the clay pipe was laid and filling was begun sometimes occurred so soon after laying pipe that the cement did not have time to set. The water then forced out a small section of cement from the joint, leaving a place for water to leak into the pipe to the basin. To repair these leaks was a difficult matter. The first one was repaired by again digging the trench and sheeting as before, but as this cost about $125 for two small leaks near each other a cheaper plan was sought. The well contractor was called upon to drive a pipe 4 to 6 ins. in diameter to the point where the leak was found. The location was determined by means of a float with candles stuck into it, the float being pushed in from the manhole by short length of ¾-in. pipe screwed together. The depth was fixed by use of the level. Care was exercised to do no injury to the clay supply pipe, in driving the wrought-iron pipe, when nearly down to the leaking joint. Within 10 or 15 minutes after the pipe was driven and the sand pumped out of it the leak was stopped by means of loose concrete and coucrete in small cloth packages rammed down the pipe.

It might be stated here, although not exactly pertinent to this paper, that water has been pumped from 5 of the 6 and 8-in. wells now in use to the extent of a 5,000,000-gal. rate per day for several hours and there was still 5 or 6 ft. of water in the basin above the bottom of the suction pipes. An estimate of the amount of water to be obtained from these wells with the present equipment gives a rate of at least 6,000,000 gals. per day, with a strong probability that 7,000,000 to 8,000,000 gals. could be obtained. Several years ago five wells, with a traction engine and a centrifugal pump placed at each, were pumped day and night for 10 days, the wells yielding 8,000,000 gals. of water per day. This, however, required drawing down the water in the wells to a depth of 5 to 7 ft. below the level of the bottom of the basin under the pumping station. This test was made during the latter part of an unusually dry summer.

In conclusion, so great are the hazards connected with quicksand excavations that under no circumstances would Mr. Brigden recommend letting a similar work by contract, either on a lump-sum bid or a price by the foot, for any attempt to make a fair proposal would be a worse risk than bucking a faro bank. If the party undertaking the work can command the needful outfit of tools and machinery, which should include light hoisting machines of some kind, such as were available on the work here described, it might be well to undertake it, as was done in this case, by day labor, but if done by a contractor it should be on a percentage basis or in some other manner which would eliminate the unjustifiable risk he would be obliged to take. It will do no harm to reiterate what has been said a number of times, that all contracts for pipe lines in wet materials, or in earth and rock, the nature of which has not been carefully investigated, is mere gambling and its natural outcome is continually coming to the surface in the shape of large losses to the contractor because of unlooked-for troubles and occasionally an unfair profit to the man who finds the conditions are better than he was lead to expect, yet this reckless bidding is going on in almost every city in the country.

General Specifications Under Which 3, 4 and 6-in. Meters of Velocity and Disk Type Are Purchased by St. Louis Water Department.

The Water Department of the city of St. Louis has drawn up sets of general specifications under which water meters are purchased. The specifications covering ⅝, ¾ and 1½-in. meters were the first ones compiled by the department and since they have not been revised recently they are omitted from the present discussion. The specifications covering the large size meters, 3, 4 and 6 in., both of the velocity and disk types, have been revised and are up to date. The present article is arranged so as not to duplicate clauses in the specifications common to the two types of meters. Each general clause is given and the stipulations peculiar to the two types are clearly indicated.

PLANS.

Each bid shall be accompanied by a detailed drawing to scale of each size and type of meter, with full description of all parts and material thereof, particular attention being paid to the cap and cover called for in these specifications.

CAPACITIES.

The capacities of the various sizes shall conform to the following list:

The capacities of the various sizes shall conform to the following list:

Nom. size. Ins.	Minimum capacity at full opening, 60 lbs. pressure. Cu. ft. per minute. Velocity type	Disk type.
3	60	45
4	110	65
6	210	120

STANDARD.

Velocity Type.—Each bidder who has not at some previous time submitted a sample of the 3-in. size of the meters to be furnished must accompany each bid with a 3-in. sample, to be delivered at the city hall, and returned at the expense of bidder. The successful bidder must submit for approval a sample of cap and cover called for in these specifications.

Disk Type.—Each bid shall be accompanied by one sample of the 3-in. size. All sample meters, except those of the successful bidder, will be returned when the contract is awarded. In the case of the successful bidder, the sample will be paid for and held as a standard of the material, design and workmanship for that type of meter furnished under these specifications. Manufacturers may submit a sample of meter for test with stock design of cap and cover. Any sample submitted under a previous specification and which is now in the custody of the meter department may be resubmitted as a sample under these specifications.

GUARANTEE.

All parts of meters furnished under these specifications which show undue wear within a period of three years from date of contract shall be supplied, within 30 days of notice, free of charge, by the firm furnishing such meters, providing the following individual meter registrations are not exceeded

Size of meter, ins.	Amount of registration, cu. ft.
3	[illegible]
4	10,000,000
6	[illegible]

Upon failure to supply meters or meter parts, as specified elsewhere, the city shall be at liberty to buy new meters on the open market and charge the cost to the firm failing to supply such meters or meter parts.

REJECTION.

Any meter furnished under these specifications not identical in design, or equal in material and workmanship to the sample either previously submitted or with bid shall be rejected. Any meter failing to pass the test for accuracy, capacity, etc., shall be rejected.

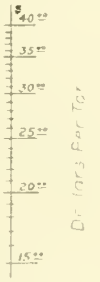
REPAIRS.

Each bidder shall furnish prices of repair parts, which prices are not to be exceeded for a period of five years from date of contract. The firm supplying meters under these specifications shall also agree to deliver parts necessary for repairs within 30 days from date of order.

GENERAL DESIGN.

Velocity Type.—All metallic parts of the gear train in contact with the water and having the same functions must be of a uniform alloy. Ample facilities shall be provided for sealing the casing and dial cap with wire car seals. Each meter furnished under these specifications shall be identified by a factory number.

Disk Type.—The hard rubber disk shall be flat and reinforced. The spindle of disk to be covered by a hard rubber roller. All metallic parts of the gear train, which are in contact with the water, must be of the same alloy. Ample facilities shall be provided for sealing the casing and dial cap with wire car seals.



Each meter furnished under these specifications shall be identified by a factory number.

CASING.

Velocity Type.—The casings of the meters furnished under these specifications may be of gray iron, and the nuts of the bolts for coupling the upper and lower casings shall be of brass.

Disk Type.—The casings of the meters furnished under these specifications shall be of brass, and the nuts of the bolts for coupling the upper and lower casing shall also be of brass. The flange of lower casing to have a recess for holding gasket.

DIAL.

Where the dial is protected by a cap separate from the casing, the cap shall be provided with a hinged plate glass cover only. Facilities shall also be provided for sealing the hinged cover with a wire car seal. The hinged glass cover shall be so constructed as to open 45 degrees. The plate glass, ¼ in. thick, shall be held in the frame of the hinged cover by means of a removable metallic ring. The dial shall be of the "straight" reading type, and register in cubic feet. The maximum and minimum registrations of the dial for various sizes of meters shall be as follows:

Size of meter, ins.	Maximum registration, cu. ft.	Minimum registration, cu. ft.
3	10,000,000	1 or 10
4 and 6	100,000,000	10 or 100

The minimum figures of registration in the above list shall also be recorded by one revolution of a hand provided for the purpose of testing the meter. Facilities shall also be provided for regearing the dials of meters to register within 1 per cent of accuracy, which in service develop any errors up to minus 15 per cent in velocity meters and 10 per cent in disk meters.

TESTS.

The Municipal Testing Laboratory of St. Louis shall determine by analysis if the composition of the several parts of the meters included in each shipment under these specifications are the same as in the sample submitted. All meters bought under these specifications shall be subjected to tests for accuracy, and shall register within the limits of accuracy as given in Table I. (Tests on stream No. 2 will be optional with the Meter Department.)

For the foregoing information we are indebted to Mr. F. L. Bock, Assistant Engineer, in charge of the distribution system.

TABLE I—ACCURACY TESTS SPECIFIED FOR 3, 4 AND 6-IN. VELOCITY AND DISK WATER METERS, ST. LOUIS, MO.

Size of meter, ins.	Rates of flow, cu. ft. per min. No. 1	No. 2	No. 3	Amount of registration in cu. ft. No. 1	No. 2	No. 3	Accuracy of registration Velocity type No. 1	No. 2	No. 3	Disk type No. 1	No. 2	No. 3
3	Full	5	2	100	10	10	98-102	95-101	85-100	98-101	98-101	95-101
4	Full	11	3	100	50	50	98-102	95-101	85-100	98-101	98-101	95-101
6	Full	21	5	100	50	50	98-102	95-101	85-100	98-101	98-101	95-101

All of the above tests will be made at pressures of about 60 lbs.

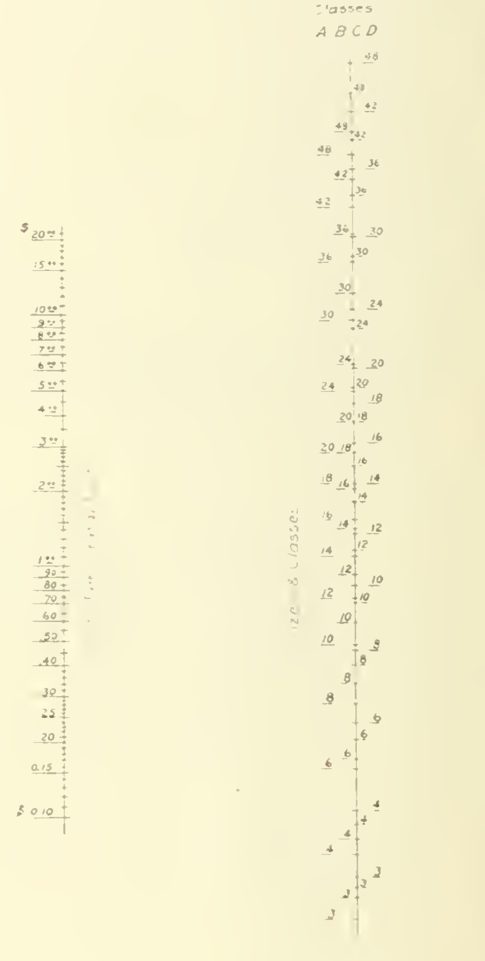


Diagram for Determining Cost of Cast Iron Pipe Per Foot from Price Per Ton and Class of Pipe.

Diagram of Costs Per Foot of Cast Iron Pipe.

Contributed by W. E. Miller, Division Engineer, Railroad Commission of Wisconsin.

Engineers, contractors and others having to do with the construction of cast iron pipe lines and their costs and values very often, if not usually, desire the cost expressed in dollars and cents per lineal foot.

As pipe of that kind is commonly bought and sold by the ton, and the price per ton varies rather widely, it has apparently been necessary, in determining the cost per foot, to make a separate calculation every time such a figure is wanted. It is also first necessary to either refer to tables of weights per foot or to have the weights memorized, as some do to an extent.

The accompanying diagram, constructed on logarithmic scales, is one that the writer has found very convenient and useful and one that would doubtless be so found by many others if brought to their attention.

A straight edge laid across the diagram so as to lie on the proper points of the outer scales will intersect the center scale at the result sought. For example: With pipe costing $26 per ton, considered as inclusive of freight and cartage if desired, 6-in. class B pipe is found by the diagram and a straight edge to cost 43 cts. per foot, while a calculation results in a slightly more accurate figure of 43.3 cts.; similarly, 12-in. class B pipe at the same cost per ton is found by the diagram and straight edge to be worth about $1.07, while a computation gives $1.0673 per foot. The classes of pipe mentioned are the American Water Works Association Standard.

Care is to be taken in using the diagram that points on the right-hand vertical scale be used for size and class of pipe—not the points under the class letters.

Similar diagrams may readily be made in the same way for other tables of weights.

Method and Cost of Laying Two 500-ft. Experimental Sections of 30-in. Steel and Ingot Iron Lockbar Pipe at Utica, N. Y.

Contributed by A. W. Peters, Engineer in Charge of Construction.

As part of the Marcy-Utica 24-in. cast-iron pipe line for the Consolidated Water Co. of Utica, N. Y., an experimental section of 500 ft. of steel pipe and 500 ft. of ingot iron pipe was laid in the Mohawk Flats. In order to provide facility of inspection this 1,000 ft. of pipe was made 30 ins. in diameter, and a manhole was provided at the junction of the steel and ingot iron sections. Both classes of pipe were of the lockbar type; length, 30 ft.; thickness, ¼ in.; to withstand a possible static head of 470 ft. The pipe was furnished by the East Jersey Pipe Co.

Both the steel and ingot iron pipe were coated at the plant by the manufacturing company with Pioneer pipe coating. In unloading these pipe from the cars a derrick and sling were used, so that the coating was uninjured; but in hauling to the job the coating was rubbed off in places, although the pipe rested upon cloth bumpers on the trucks. The spots were retouched before laying with a paint furnished by the pipe company.

The excavation, pumping and backfilling were done by the contractor having the whole extension in hand, but the laying, riveting and caulking were sublet at a price of 70 cents per foot to a contractor experienced in steel pipe work.

The costs as shown are derived from the actual field costs, and so in applying to future work the necessary additions, such as contractor's profit, overhead charges, etc., should be made.

Because of the location of this experimental section—in an absolutely flat and level country, affording little runoff for the heavy rainfalls which occurred during the course of the work—the costs are high, due chiefly to the excessive amount of pumping required and delays incidental thereto during laying, riveting and caulking.

Excavation.—The material encountered was a brownish loamy clay, which was heavy and sticky to handle, due to the wet conditions of the trench. The sides of the ditch were slightly sloped and stood without sheeting during the month of bad weather in which this ditch was open. The average depth of ditch was 5 ft.

The ditch was dug with an Austin excavator working under the most adverse conditions. It was at all times necessary to support the machine on planking, and even with these precautions the machine was nearly "mired" several times. The bell holes were dug by hand, one every 30 ft. Three gasoline diaphragm and two hand diaphragm pumps were used.

Placing and Riveting.—Pipe laying started on Oct. 1, 1913, and from the start the ditch was very wet. In places streams from ½ in. to 1 in. in diameter occurred in the sides of the ditch, due undoubtedly to channels left by decayed organic matter. These conditions were the result of a rainfall of 2¾ ins. on Sept. 21. The general contractor then placed three gasoline pumps and two hand pumps on the job.

The placing and riveting of this 1,000 ft. of pipe took from Oct. 1 to Oct. 16, during which period no time was lost because of bad weather, but only slow progress could be made, due to the large amount of ground water.

The maximum length of pipe laid and riveted per day was 240 ft., or 8 joints. The minimum length of pipe laid and riveted per day day was 30 ft., or 1 joint. The average amount of pipe laid and riveted per day was 90 ft., or 3 joints.

Caulking.—On Oct. 18, when the sub-contractor was ready to begin caulking, a further rainfall of 1¾ ins. occurred, followed by light showers on the 19th and the 20th. Caulking was started at noon on the 21st. Further showers occurred on the 24th, 25th and 26th, but caulking was continued with all pumps working.

The maximum number of joints caulked per day under these conditions was six. It should be noted that because of the local conditions the bell holes were pumped twice—once for riveting and once for caulking. If leaks develop or show up during the test of this section some bell holes will have to be pumped for the third time.

It is evident that in a more hilly country, with natural drainage, the pumping charges would not be nearly so great as in the present case under the same bad weather conditions.

COST DATA.

Cost of Labor:

Items—	Total cost.	Cost per ft.
Unloading	$ 55.00	$0.055
Hauling	115.14	0.115
Excavation (ditch, Austin mach.)	448.90	0.449
Excavation (Bell holes, hand)	99.24	0.099
Pumping	401.00	0.401
Placing	291.18	0.291
Riveting	282.75	0.283
Caulking	176.25	0.176
Backfill	100.02	0.100
Total cost for labor	$1,969.48	$1.969
Cost of Material:		
Steel pipe	$2,000.00	$4.00
Ingot iron	2,450.00	4.90
Cost of Labor and Material:		
Steel pipe		$5.97
Ingot iron pipe		6.87

Particular attention is directed to the fact that the foregoing costs do not include engineering, overhead charges or contractor's profit.

Bridges

Design Features of the Reinforced Concrete Viaduct on Hopple St., Cincinnati, Ohio.

(Staff Article.)

In our issue of Feb. 11, 1914, we discussed the general features of the Hopple St. Viaduct in Cincinnati, Ohio, and gave in detail the principal design features of the east and west reinforced concrete retaining wall approaches. The following article will treat of the design features of the superstructure of this viaduct which consists of a series of reinforced concrete spans, each span comprising two reinforced concrete cantilever arms supported on reinforced concrete piers. We shall discuss the design features of the piers in a subsequent issue.

II. Superstructure.

The main structure consists of 25 spans on skewed piers, each span consisting of two curved cantilever arms and a single cantilever arm at each end of the viaduct. Beginning at the east end (the viaduct being on an east-and-west street) the lengths of the cantilever arms of each span are as follows: One cantilever arm, 35 ft.; span No. 1, 35 and 40 ft.; No. 2, two 40-ft. arms; No. 3, 40 and 35 ft.; No. 4, 35 and 30 ft.; No. 5, 30 and 35 ft.; No. 6, two 35-ft. arms; No. 7, 35 and 40 ft.; Nos. 8 to 12, inclusive, two 40-ft. arms; No. 13, 40 and 35 ft.; Nos. 14 and 21, inclusive, two 35-ft. arms; No. 22, 35 and 40-ft. arms; Nos. 23, 24 and 25, two 40 ft. arms; and one 40-ft. cantilever arm. The viaduct has a total length of 1,930 ft. and an extreme width of 60 ft.

LOADS AND ALLOWABLE STRESSES.

Dead and Live Loads.—In estimating the dead load of the structure, reinforced concrete was taken at 150 lbs. per cubic foot, and the wood block paving and sand cushion at 30 lbs. per square foot.

The live load assumed for the roadway slab was 600 lbs. per square foot; for the sidewalk slab, 100 lbs. per square foot; and for the cross beams, a 20-ton road roller plus 20 per cent for impact, two-thirds of the load on the rear axle and one-third on the front axle. The live load assumed for the inside cantilever ribs consisted of a 40-ton double-truck electric car plus 25 per cent for impact, and 150 lbs. per square foot on an 8-ft. width of roadway adjacent to the curbs. The live load on the outside cantilever ribs was taken at 150 lbs. per square foot on the roadway and 100 lbs. per square foot on the sidewalks.

Allowable Stresses.—The following allowable stresses were used in designing the superstructure: Tension in reinforcing steel, 18,000 lbs. per square inch; compression in concrete, on extreme fiber, 700 lbs. per square inch; and shear in concrete, 65 lbs. per square inch.

ASSUMPTIONS AS TO THE DESIGN.

In designing the viaduct spans each pier with its adjacent cantilevers is considered as a unit. No bending or arch action is transmitted from one pier to another, shear only being transmitted by the joint between the ends of adjacent cantilevers.

Since each pier and its cantilever beams are symmetrical about the center line of the pier there is no overturning moment in the pier due to dead or to full live load on the cantilever arms.

SPECIFICATIONS FOR SUPERSTRUCTURE.

The following extracts have been made from the specifications covering the construction of this viaduct:

If the whole width of the viaduct, including the four ribs and the sidewalks, can not be concreted continuously work must be divided so that each rib shall be concreted continuously from expansion joint to expansion joint. All concrete shall consist of a 1:2:4 mixture, using crushed stone or gravel 1 in. or under. Forms shall, in general, not be removed in less than 7 days for column and beam sides, 10 days for slab forms, and 21 days for beam and girder bottoms, except for vertical surface carrying no load. For work done in winter forms shall not be removed in less than 1½ times the above periods. The contractor must use steel centering for the main cantilever beams having curved bottoms. Steel centers must extend from pier to pier, as no shoring to the ground between piers will be permitted. Centers shall be built to allow for deflection. Horizontal reinforcing bars must be of single length between supports. All bars in the top of beams, girders and slabs over supports shall extend back from the face of the support a distance of at least 40 diameters of the bar, and in the bottom a distance of at least

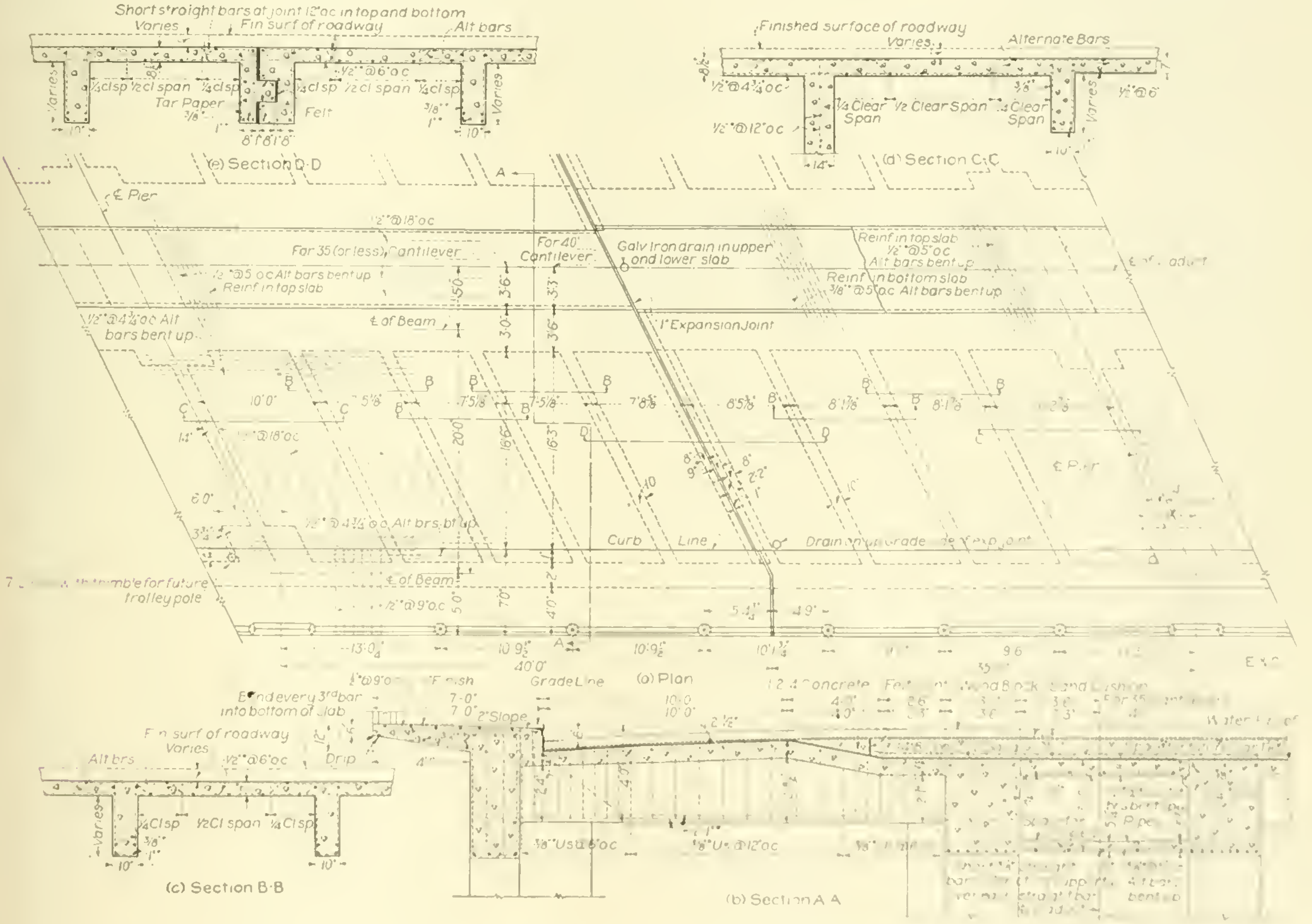


Fig. 1. Plan, Cross Section and Longitudinal Sections of Hopple St. Viaduct, Cincinnati, O., Showing Details of Design.

20 diameters, in no case less than 12 ins. The ends must be bent in the form of a hook, and a stub must be inserted in it if the full grip as specified can not be obtained. The thickness of concrete outside of the reinforcing steel shall be: in piers, 2½ ins.; in beams, girders and walls, 1½ ins.; in slabs, ¾ in. between the bottom of slab and the main reinforcing bars.

The entire top of the concrete slab of the roadway is to be waterproofed as follows: The surface is to be given a primary coat of pure asphaltum thinned with 62° benzine after being thoroughly cleaned. Immediately after applying the primary coat there shall be a coat of refined asphalt applied hot. Into this, while hot, embed a layer of heavy burlap from which the water has been removed, and which has been previously saturated and coated with pure asphaltum; the burlap used to weigh approximately 8 oz. per square yard. On this apply another coat of hot asphalt into which, while hot, apply a layer of best quality wool felt, saturated and coated with asphaltum weighing 11 lbs. per 100 sq. ft. On top of this layer of felt apply another layer of hot asphalt. All burlap and felt must be bedded into the asphalt while hot and in no place shall the burlap or felt touch each other. At joints the layers shall lap 6 ins. in the direction of the grade. The waterproofing shall be flashed up on the curb into the "V" groove to be made in the concrete and around other objects.

The structure in the center of each span shall be distinctly divided by a joint as shown on the detailed plans. In making this joint the concrete on one side shall be placed first. A layer of felt 1 in. thick, saturated with asphalt, shall be placed against all vertical surfaces of the concrete. A layer of wool felt such as is used in the waterproofing for the roadway slab then shall be placed over this felt and over all horizontal surfaces of the concrete. The concrete on the side of the joint not yet in place then shall be placed directly against the layer of wool felt.

DESIGN FEATURES OF CANTILEVER ARMS.

Each cantilever arm comprises four curved reinforced concrete ribs which are designed as cantilevers from the skewed piers. The two inside ribs are spaced 10 ft. on centers (5 ft. on each side of the center line of the viaduct). The distance between the inside and outside ribs is 20 ft. on centers. Figure 1(a) shows a part plan of a span consisting of a 35 and a 40-ft. cantilever arm, and indicates the type of construction used. The portion of the viaduct which will be considered in this article under the heading "superstructure" is that part above the spring line of the curved cantilever ribs.

The floor system of the 40-ft. clear roadway is of the beam-and-slab construction, the beams extending from the inside to the outside ribs. Reinforced concrete slabs are constructed between the inside ribs, no beams being required on account of the small span (see Fig. 1, a). Figure 1 (b) shows a cross-section of the roadway of a 40 ft. cantilever arm. It will be noted that provision was made for future street car tracks, although none are to be provided at the present time. For this purpose the floor construction near the center of the roadway is depressed, the space being filled with a 2-in. layer of sand and a layer of 1:4:8 concrete. An inclosed space for pipes is provided under the floor slab between the inside cantilever ribs. The bottom of this compartment consists of a reinforced concrete slab built monolithic with the ribs, and the top consists of a detachable reinforced concrete slab supported by brackets on the ribs. The cross-beams have a variable depth, and are reinforced as shown in Fig. 1 (b). A longitudinal section through two intermediate cross-beams, indicating the reinforcement for the beams and the included floor slab, is shown in Fig. 1 (c). Figure 1 (d) shows a section through the beam on the center line of the pier and the adjacent cross-beam, and Fig. 1 (e) shows a longitudinal section through the joint between the two cantilever arms. The latter drawing shows the details of this joint. It will be noted that the joint is so constructed that shear is transmitted by it. These sections show the thickness of the floor slab at various points.

The 7-ft. sidewalk on each side of the roadway extends 1 ft. inside of the outside rib, over the rib, and cantilevers 4 ft. outside of it. Figure 1 (b) shows a section through the sidewalk. This drawing gives the principal dimensions and indicates the size and arrangement of the reinforcement.

The cantilever ribs have spans of 30, 35 and 40 ft. depending upon their position in the viaduct. Figure 2 (a) shows an elevation of the outside rib for a 40 ft. cantilever, and Fig. 2 (b) shows a cross-section of this rib. Figure 2 (c) shows an elevation of the inside ribs for a 40-ft. cantilever, and Fig. 2 (d) shows a cross-section of this rib. The thickness of the outside rib is 2 ft. 0 ins., and that of the inside rib, 3 ft. 6 ins. The bottoms of these ribs are curved, except for a 1-ft. 6-in. section at their outer ends, giving the structure the appearance of a series of arches. The radius of curvature for the outside rib is 52 ft. 2 ins., and that of the inside rib is 34 ft. 1¾ ins. The depth of the outside rib (including the floor slab) at the end of the cantilever is 3 ft. 9 ins., and at the pier its depth is 11 ft. 9 ins.; the corresponding dimensions for the inside rib are 4 ft. 0 ins. and 11 ft. 6 ins. Details of the 1-in. expansion joint at the end of the cantilevers are shown in Figs. 2 (a) and 2 (e). The drawings show the principal dimensions of the ribs and the size and arrangement of the reinforcing rods.

The rods in the tops of the ribs are placed in three layers. The distance between the layers of rods is 4 ins., the top layer being 3 ins. from the surface. To insure an accurate spacing of these rods angle spacers are specified. Figure 2 (e) shows a detail of the spacers for the inside ribs.

The reinforcing rods in the bottoms of the ribs are spliced near the spring line. The splice is made by lapping the bars and fastening them together with two clips. A detail of these clips is shown in Fig. 2 (f).

A 3x2½x⅜-in. curb angle is anchored into the concrete by means of anchors consisting of 1½x3/16-in. bent bars, spaced 2 ft. on centers and staggered. Figure 2 (g) shows a detail of the construction at the curb.

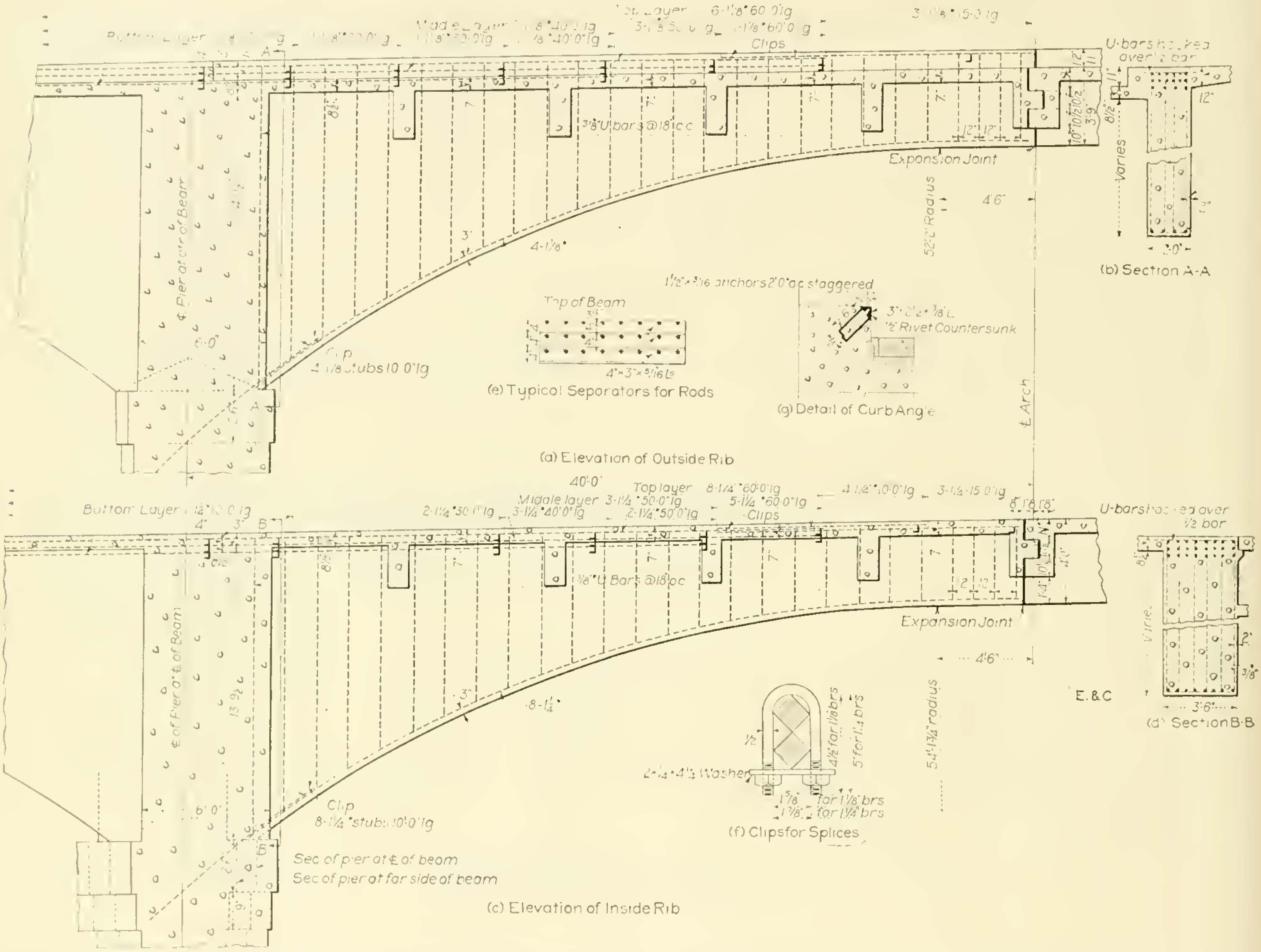


Fig. 2. Elevations and Sections of Outside and Inside Ribs of Superstructure and Some Other Details of Hopple St. Viaduct.

REINFORCED CONCRETE RAILING

The cantilevered sidewalk carries an ornamental reinforced concrete railing. The railing extends 3 ft. 4 ins. above the sidewalk slab, and is held in place by hexagonal reinforced concrete posts. Figure 3 (a) shows a detail of a portion of this railing, and indicates the type of construction used. Figure 3 (b) shows a cross-section of the railing and an elevation of the post, and Fig. 3 (c) shows a cross-section of the hexagonal post. These drawings show the dimensions, and the size and arrangement of the reinforcing rods. It will be noted that the slabs of the railing are cast separately from the posts, and that they fit into grooves in the posts and sidewalk slab. The slabs are 6 ins. thick, except at the paneled portion, where the thickness is 4 ins. Figure 3 (d) shows a horizontal section through the railing slab at an expansion joint. In casting the slabs adjacent to an expansion joint three holes, each 2 ins. in diameter, are left at the locations of the vertical reinforcing rods. These holes are to be filled with grout after the slabs are in place. The 1-in. expansion joint is filled with tar paper and felt.

QUANTITIES OF MATERIALS AND COST.

The viaduct will have a sand blast finish. The approximate quantities of materials in the superstructure are: Reinforced concrete, 11,600 cu. yds.; concrete railing, 4,100 lin. ft.; waterproofing, 10,200 sq. yds.; concrete ballast for future street car tracks, 1,000 cu. yds.; and curb angles, 13 tons. The drainage and stairway are also included in the superstructure. The contract price for the superstructure is $189,000.

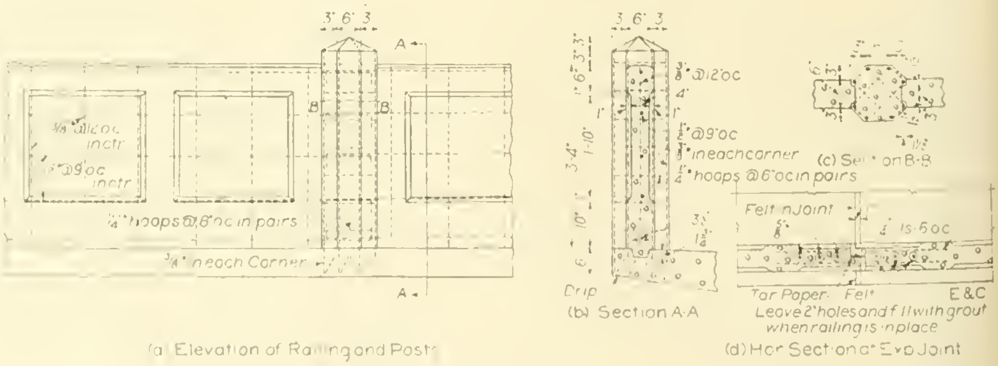


Fig. 3. Elevation and Sections of Reinforced Concrete Railing of Hopple St. Viaduct.

The Hopple St. Viaduct was designed under the direction of Mr. H. M. Waite, chief engineer, and Mr. F. L. Raschig, engineer of structures, Department of Public Service of Cincinnati. The construction will be continued under Mr. F. S. Krug, who has succeeded Mr. Waite as chief engineer, and Mr. F. L. Raschig.

The Montana Institute of Municipal Engineers at their recent annual meeting elected the following officers and directors: President, F. C. Snow, Bozeman; vice president, C. W. Helmick, Helena; secretary-treasurer, C. C. Widener, Bozeman. These officers with the following are the board of directors: Wm. M. McLean, Lewistown; M. L. Morris, Great Falls.

A Discussion of the Advantages and Disadvantages of Fixed and Three-Hinged Concrete Arches, With Solid and Ribbed Soffits and With Solid and Divided Piers.

In our issue of Nov. 26, 1913, we printed an article on the relative advantages of fixed and three-hinged concrete arches. This article gave the chief advantages of the three-hinged type, and also discussed the advantages possessed by arches with two ribs over those with solid soffits. A discussion was also given concerning the relative efficiency of solid and divided piers. The conclusions drawn by the writers of this paper are that three-hinged concrete arches are more economical and reliable than those with no hinges and that concrete arches with two separate ribs are more economical than those with solid soffits when the span in each case is greater than 100 ft. The following discussions of this paper, which were abstracted from Vol. XL., p. 131; Proceedings, American Society of Civil Engineers, are herewith given, as the importance of the subject warrants full discussion:

DISCUSSION BY C. E. GREGORY.

As designing engineer in charge of the design of the bridges built by the Board of Water Supply and referred to in the paper under consideration the writer wishes to make a few statements to amplify the relation of the work as done by the Board of Water Supply to this paper, and to put others who were concerned in the design of these bridges in a right position.

The authors reach the conclusion that in practically all cases the three-hinged arch is more economical than the ordinary fixed-end arch. When designing the Traver Hollow Bridge the writer did not, and does not now, concur in such a conclusion. The hinged type was selected largely because of an expected slight settlement of the foundations, rather than because of the economy of this type. The preliminary estimates indicated that a three-hinged arch would be more expensive than a fixed arch.

The economy of the divided pier is brought out by the authors, but the manner in which the design of the piers of the Rye Outlet Bridge was arrived at differed considerably from that described in the paper, although meeting the assumptions of the authors. The interesting and unusual feature of this bridge is that it is an arcade of five reinforced concrete arches supported on exceptionally high piers. It was found that the yielding of the top of a high pier of ordinary shape under the unbalanced live load arch thrust (the height of the highest pier in this bridge being a little more than 100 ft.) would be sufficient to create considerable stress in the arch without exceeding the usual limits of stress or without creating tension in the pier. The design was made to limit the yielding of the piers to that which could be safely and economically provided for in the arches. The entire arcade was treated as a complete elastic structure, and determination was made of what proportion of the unbalanced thrust from a loaded arch would be carried by the adjacent pier and by other arches and piers of the arcade. The shape and size of pier adopted was found to be necessary from calculations made as just outlined.

DISCUSSION BY HENRY H. QUIMBY.

Experience in the design of concrete arches confirms the opinion expressed by Mr. Gregory regarding the relative economy of fixed and hinged rings, and the justification for the use of hinges. Computation for comparison of designs in several cases have failed to show any saving in cost of a three-hinged type over a fixed type when the ratio of rise to span was fairly good. Hinges are justified only when the foundations are unsatisfactory, or in cases where the arch is very flat. In a flat arch the crown movement due to change in length of ring from temperature is greater than in a high-pitched one, and the temperature stress may thus become great enough to warrant the use of a hinge to avoid it. The arches shown in the paper, however, are not flat, but have a good rise.

The authors give estimated quantities of concrete required for the arch rings in the two designs shown, and represent the fixed type as requiring 90 per cent more concrete than the hinged. This is hard to understand. A fixed ring should require only a little more than a hinged ring. The depth at the crown must be about the same for each because the horizontal thrust is about the same, and any theoretical eccentricity, due to unsymmetrical loading, in the fixed ring will probably be fully matched by the effect of friction of the hinge in the other case, for the inevitable deflection caused by the weight of the floor imposed after the closure of the arch will be likely to produce the effect of eccentric thrust. At the quarter points the hinged ring will need considerably more depth than the fixed ring, and it will be only at the springing line that the fixed ring will be deeper and require more concrete. As the cost of effective hinges is considerable, the net result is generally the reverse of that reported in the paper.

If it be said that the greater depth of the fixed ring design in the paper was made because of temperature stress, the reply is that temperature stress necessarily increases with the depth of the ring, and adding to the depth because of such stress is somewhat like piling a load on a horse's back to enable him to pull a heavier wagon—the poor legs may be overtaxed. It would seem, therefore, that there must be something wrong either with the estimates or with one of the designs.

The stone hinges shown in the paper give the impression of being likely to develop friction if they work at all, and this will entail eccentricity of stress in the arch ring, to that extent detracting from the only advantage of the hinge.

The matter of appearance of an arch is important in many places, and the bulging at the quarter that is characteristic of the ordinary hinged ring is distinctly ungraceful and displeasing to the miscellaneous eye which cannot understand the reason for it, while the eye of the designer who knows that the point of greatest shear is the point of greatest single stress, desires to see the arch thickest at the spring where that stress is the greatest.

One very important consideration in a discussion of stress in a masonry arch is the method of construction, and it is generally far more vital than temperature fluctuations. The old-fashioned method of laying up an arch ring from the springing line to the crown, with the attendant progressive deformation of the centering and consequent cracking of the earlier portion of the ring practically reduces the working thickness of the ring to a fraction of that designed, and may be the real cause of some failures that have occurred. The proper way to avoid cracks or initial stresses in the arch ring due to settlement or compression of the centering during construction is to build the ring in voussoir blocks with narrow key spaces between them. Then when all the blocks are made, and the centering has, therefore, nearly its full load, after the blocks have seasoned and shrunk, cast the keys. Of course, if the arch is small enough to permit the casting of the full length of the ring in one day—say within ten hours—the separation into blocks will not be needed, because the first portion cast will be still plastic enough when the ring is closed to adjust itself to the deformed centering. It is impracticable to eliminate initial stress wholly, for any of the dead load that is added subsequently to keying will cause some deformation of the ring and consequently initial stress somewhere, but this should never be enough to be serious.

Another point in the paper that seems not entirely clear is the claim of marked advantage in the I shaped arch rib. The arch ring is a column—not a beam. If the curve is true and the line of thrust central there is absolutely no beam action. Of course the width (horizontal thickness) of the rib and its depth (vertical thickness) should both be theoretically adjusted to make a column that will fail by crushing rather than by buckling, and this will require some minimum width. As regards lateral buckling, any given width is as effective at the mid-depth as at the flanges, and in any case suitable transverse bracing should be provided. The method of estimating the saving due to the coring out of the concrete at the sides is hardly fair, because a very large part of the cost of such concrete is in the support of it, and any increase or decrease in quantity will properly be charged for at only the cost of the material and the placing of it. Besides, the extra cost of the additional formwork required for the paneling will materially reduce the seeming economy in material.

DISCUSSION BY A. C. JANNI.

The following comments were abstracted from the discussion of A. C. Janni:

Although the indeterminations arising from friction on the hinges and from other causes do not prevent a designer from patching up some kind of a design for a three-hinged arch, it is not possible to accept statements as to the great reliability of such a design.

However, the hinges in a concrete or masonry arch can sometimes be used to advantage during the construction of the arches, when there is no steel reinforcement to take up the tension at the spring line and at the key due to a possible excessive settlement of the hinges; then after the centering has been removed the hinges can be connected. Thus the arch behaves as a three-hinged arch as far as the dead load is concerned, and as a rigid arch for the live load and for temperature changes. In that case the hinges, free from the enormous pressure of the dead load, can easily accompany any movement, being concreted afterward when their behavior begins to be extremely doubtful.

The granite railroad bridge at Morbegno, Italy, with a 229.65-ft. span and a 32.8-ft. rise, has been built in accordance with the foregoing method.

Concerning the statement that the trial design for a fixed-end arch takes several days for each analysis it can be said that it takes no longer than for a three-hinged arch, and possibly less time. The stresses in an arch with fixed ends due to dead load and temperature can easily be calculated in a day.

A new method of designing an arch, based on the rather recent theory of the "Ellipse of Elasticity," is described in a paper by the writer in the Journal of the Western Society of Engineers, May, 1913. This method is purely graphical and holds, irrespective of the geometrical form of the axis of the arch, irrespective also of the law of variation of the cross-section of the latter, and of the assumptions of loading. In other words, it is possible by this method to draw the lines of influence for moments for any section of the arch before the loads are applied on the arch, and therefore the most prejudicial hypothesis of loading for the chosen section is given at once, without any trial. The paper also shows a practical application of this method to the design of an arch.

In contrast with the unreliability of the results in a design for a three-hinged arch, it is not without interest to relate that for a fixed end reinforced concrete arch, built in France with a span of 318 ft., the difference between the stresses calculated and the actual stresses reached a maximum of 11.7 per cent.

Every engineer fully appreciates the importance of these results, especially when he considers that in steel bridges of the best design that difference reaches 25 per cent, and when designed on the assumption that there is no friction at the joints the difference may be more than 100 per cent.

The horizontal thrust, due to temperature changes, is directly proportional to the coefficient of expansion, to the change of temperature, to the modulus of elasticity of the material, and to the span (measured at the spring points of the geometrical axis of the arch); but it is inversely proportional to the moment of inertia of the whole elastic arch with respect to the horizontal (if the arch has its spring points at the same level) passing through a certain point called the center of gravity of the elastic arch.

If, for the same span, arches with different rises are drawn, it can be seen that, although the factors directly proportional to the horizontal thrust remain unchanged (the variation of the span of the geometrical axis may be disregarded), the moment of inertia of the arch increases rapidly with the increase of the rise of the arch; so that the effects of a change in temperature diminish in importance with the increase of the rise.

The shrinkage of the concrete due to setting and that due to the dead load are two important causes of stress to be taken care of, especially the former. If the arch is very flat and has a large span, the best and most economical way to take care of the shrinkage due to setting is by neutralizing its effects, as much as possible. A span of 170 ft. with a 17-ft. rise, and two spans of 140 ft. with a 14-ft. rise, both designed by the writer, have been built by breaking the arches into voussoirs each about 20 ft. long and leaving between each two consecutive voussoirs a small 2-ft. voussoir, or key. The voussoirs were cast first, and the keys were not cast until the last concrete of the voussoirs was ten days old, thus avoiding, for the 170-ft. span, important tensile stresses in the intrados at the key and in the extrados at the springing line, due to a shortening of ⅞ in. of the geometrical axis of the arch.

If, as the authors state, an average pressure of 12,000 lbs. per square inch on the crown hinge of that arch was calculated, it must be inferred that the maximum pressure on the hinge was about 21,000 lbs. per square inch at the theoretical point of contact with its groove, and it may be assumed that it was reduced to zero at each end of the groove. The question then arises: At such a vital point of a bridge should the material be submitted to stresses which are never allowed for such material at other much less important points? The inadvisability of trusting to a 2½-in. pin the stability of such a large mass of material is another important factor. In dealing with such pressures, however, there is no longer any question of the possibility of assuming that at that point there is a hinge, ready to work; it is no longer a matter of design, but of guesswork. When a hinge of such small diameter is placed at the key, a malicious attempt to wreck the span could be successfully carried out.

Another kind of hinge—fortunately very seldom used—is a ball-and-socket joint of cast steel consisting of a hemispherical protuberance fitting into a hemispherical cavity. Any horizontal displacement of the crown of the arch in a structure of this kind cannot be contemplated seriously; there is no reason for using such a hinge. In the construction of a three-hinged arch, the use of a ball-and-socket hinge means simply the addition of more friction at that joint.

Concerning the saving of material by building the ribs of an I-shaped section, it may be remarked that such a saving, in a general way, does not necessarily mean a saving of money. The three-hinged arch, on account of its lack of rigidity, requires a stronger system of stiffeners than that needed by a fixed end arch, and the quantity of concrete in the stiffeners for the former must, necessarily, be greater than in those for the latter; therefore the statement advanced by the authors, that the quantity of concrete was the same in each case, cannot be accepted.

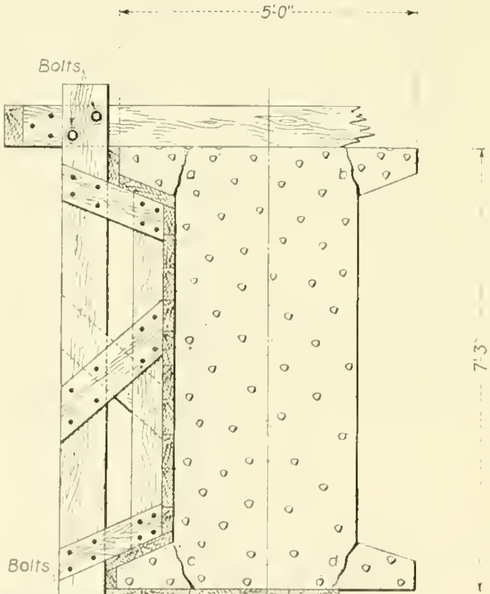
In conclusion, it is to be noted that the statement concerning the complication of design for a fixed-end arch and the case with which such an arch can be upset by a slight movement of one of its supports, is not altogether justified. By the theory of "virtual work" or more elegantly and speedily, by the method of the "ellipse of elasticity," the designer can easily take care of the displacement of the supports, and even of a yielding of the soil on which the footings rest.

There is no doubt that the design for an arch is one of the most delicate problems of engineering; but, after all, it may be done without any great trouble if the designer is sufficiently familiar with his work.

The theories used are not monopolized by a few persons, but are quite accessible to any one of training and application. Fortunately for the designer, those theories cannot be reduced to "rule-of-thumb," for several reasons. If an engineer, satisfied with what he was taught at school, does not keep in touch with the daily progress of science, he runs the risk, nowadays, of being behind the times. If, for example, in designing an arch he still assumes first that the arch is entirely loaded and then that it is half loaded and half unloaded, still believing that in this way he can determine the maximum stresses in that arch he is wrong; those assumptions were made several years ago, because there was then no means of ascertaining, conveniently, the most prejudicial hypothesis of loading; at present, however, that method of computing stresses is antiquated and inadequate.

DISCUSSION BY PHILIP AYLETT.

In America, the ribbed arch, especially the three-hinged type, has not received that recognition of its merits which it deserves. Its economy and advantages are better known and more highly appreciated in Europe, where it has been used extensively under almost all conditions of site and traffic, and in great variety of form and design.



Forms in 6' to 10' longitudinal sections, each section detached by removal of bolts in verticals.

Fig. 1. Sctional Form for I-Section Ribs of Concrete Arches.

Comparatively few concrete bridges of the rib type proper (hinged or fixed) have been built in America, and these only within the past decade. The literature on the subject, therefore, is comparatively scant.

The authors of this paper deserve the thanks of all interested in the use and development of this type in America, where its future is so promising and its field so extensive.

The use of three-hinged arches on compressible material is fully justified by ample experience and precedents. The foundations of abutments and piers in such cases, however, demand more careful investigation study, and design. A fixed or a two-hinged arch may also often be built with entire safety on compressible material, provided the foundations are properly designed and built, including the use of batter piles and other methods of taking up the thrust.

The seeming economy of the rib of I-section, as compared with the rectangular section, will not show so vividly in practice as on paper. This saving in concrete in the I-section will be largely if not entirely offset by the greater expense caused by the additional material and labor involved in making and placing the I-forms and in depositing the concrete therein. Forms for such sections also necessitate the cutting up of the lumber into many short lengths, thus making much of the material unfit for other use. For example, Fig. 1 shows an I-section form designed for the purpose of economizing in material and in the labor of making, placing and dismantling it. This form is also capable of repeated use, as it is built in sections with this end in view. Yet the additional material and labor involved, in comparison with forms of rectangular section, is apparent at first glance.

In addition, the I-section rib invites and makes possible injury and defects to the permanent structure from which the rectangular section is exempt. No one can foresee or forestall settlements in falsework, or centering from timber shrinkage, "taking up," "biting," columnar shortening, foundation movements, expansion, contraction, etc., falsework or centering moves upward as well as downward on account of the unsymmetrical placing of the load, or in cases where the falsework becomes wet after being thoroughly dried out. A rectangular arch rib is capable of sustaining more settlement or upward movement of the falsework without injury than is possible with an I-section, because of the projecting flanges of the latter. Settlements or movements of the falsework prior to the removal of the I-forms seriously endanger these concrete flanges, and will probably cause rupture at or near their junction with the body or web of the section, as at *a*, *b*, *c* and *d* (see Fig. 1).

From a number of cases it has been observed that the maximum movements of falsework (both up and down) occur during or immediately after the construction of the arch ribs (except in cases of accidents, etc.), because this is the period of the initial adaptation or accommodation of the falsework to its superimposed load. During this period, therefore, the I-forms are always found in position and cannot be dispensed with. The danger of flange rupture is most imminent during the initial stages of arch action, that is, when arch action begins to assert itself, the concrete being comparatively green.

Flange rupture may occur, however, after the rib has passed into the self-supporting stage, or even before arch action asserts itself at all. For example, if a downward movement of the falsework occurs just as arch action begins to take place, the rib offers some resistance to the movement, although it may be incapable of entire self-support. The result is rupture in the bottom flanges and cracks opening upward, as at *d* (see Fig. 1).

If an upward movement of the falsework occurs (on account of swelling, distortion or accumulation of drift and debris against the forms), flange rupture takes place in the opposite direction, the cracks opening downward, and both top and bottom flanges may be affected, as at *a*, *b*, *c* and *d* (see Fig. 1). Flange rupture prior to arch action is most likely to occur in connection with the use of heavy reinforcing members, such as I-beams, lattice girders, etc., as these by their own arch action are capable of resisting the movements of the falsework.

It must also be remembered that at or near the points *c* and *d* in the I-section the poorest and weakest concrete will undoubtedly be found. It is here that the greatest difficulty will be met in securing good concrete, on account of the re-entrant angles in the forms, and the proximity of the reinforcing members thereto. Hence the flanges which must resist the greater stress from such movements are less capable of doing so, on account of the quantity and quality of the material.

The likelihood of flange rupture, especially after arch action in the rib, may be decreased somewhat by avoiding a rigid connection between the sides of the I-forms and the supporting timbers below. Thus (see Fig. 1) such a precaution may be provided by simply omitting, on both sides, the lower bolts shown in the verticals, and using stop-blocks instead. Slip-joints thus formed would permit the downward movement of the falsework without injury to the flanges, provided the arch had attained the stage of self-support.

Although the authors affirm that "the shape of the piers" is a "most important point in the design," they appear to overlook this and other important points relating to the design and construction of bridge piers, especially in flowing streams.

In the comparative estimates of the braced and solid types of piers, apparently, only the difference in concrete has been considered. The greater length of cofferdam required by a pier of the braced type (with or without diaphragm or foundation offset) should be considered. In constructing cofferdams in flowing streams the usually important thing is time, for more risk is attached to cofferdam building in flowing streams than in any other operation connected with ordinary bridge building, especially where there is ice or logs of wood moving rapidly.

The many-sided dam of the braced type pier requires much maneuvering of the pile-driver in order to effect the frequent changes of direction. This involves change of anchorage, backing, going forward, etc., all by men who are not sailors, while breasting a current and with risk of disaster by flood.

The solid rectangular pier requires less cofferdam, less maneuvering of the pile-driver, less exposure to flood at the most critical time, and consequently much less time and expense. These are among the important points which should be considered in pier design and construction, and cannot be overlooked in any estimates.

There are, however, other features of the authors' braced pier which appear so serious as to preclude entirely this type from consideration in streams of the character mentioned.

The authors state: "This type (braced) of pier is especially economical for a bridge in running water in which there is likely to be heavy blocks of ice or logs of wood moving rapidly."

The braced type of pier is exposed, not only to the accumulation of drift on its upstream end as in solid piers, but will also catch and hold logs and drift in the space between the legs on both sides. The bouyancy and enormous leverage of a mass of logs and drift would exert a simultaneous lifting and wrenching action on the pier, increasing with the pressure of the rising flood. Masses of drift cause scour around the piers, and this would probably extend below the diaphragm if its depth extends only to the point where the pier legs enter the ground. It is difficult to reconcile the obstructive braced pier with the nation-wide conservation movement of today, especially in the face of the experience of the West and Middle West in 1913. Amid the losses of lives and property the bridge engineer's lesson stood out above all others—unmistakable and specific. It was "Conservancy of Rivers" by

(a) Avoiding unduly wide and obstructive piers;

(b) Reduction in the number of piers by an increase in the span lengths, and

(c) Increasing the floor level elevation.

DISCUSSION BY L. J. MENSCH

The statement of the authors that three-hinged arches contain only about half as much masonry as properly designed fixed arches is only true for bridges of very small rise, and the writer would be interested to see the calculations for the bridges shown in the original paper, as his own tables do not show such a difference. The fact is that the thrust in both types is practically the same. The bending moment caused by a concentrated load in a three-hinged arch is greatest when the load is 0.211 l from the abutment, and its value is about $P \times \frac{l}{10}$; in a fixed arch a concentrated load produces the greatest bending moment when placed 0.3 l from the abutment, and its value is about $P \times \frac{l}{16}$.

It is customary to calculate a bridge for a uniform load, w, placed on its half span. The greatest bending produced in a three-hinged arch occurs at the quarter span, and amounts to $\frac{w \times l^2}{64}$; in a fixed arch it occurs at the point 0.3 l from the abutment, and amounts to $\frac{w\, l^2}{120}$; at the same time the moment at the abutments is $\frac{w\, l^2}{64}$.

There is nothing to justify the statement that there is a considerable saving in materials in a three-hinged arch over a fixed arch, unless the temperature stresses and the stresses due to the shortening of the arch and the shrinkage of the concrete play an important role, which is only the case in arches of comparatively small rise and large depth. Professor Howe, in his treatise on arches, showed that the thrust from a change of temperature in a parabolic arch of constant $\frac{d\,s}{I}$ may be found by the formula, $\frac{45}{4}\,\frac{E\,I\,c\,t}{f^2}$, where E = the modulus of elasticity, I = the moment of inertia of the crown section, c = coefficient of expansion, t = change of temperature, in degrees Fahrenheit, and f = rise of arch; and that its point of application should be assumed at $\frac{1}{3} f$ below the crown.

The writer finds that in a parabolic arch, as usually designed, the thickness of the arch increasing at the abutment to about 1½ to 2 times the crown thickness, the thrust may be found by the formula, $24\,\frac{E\,I\,c\,t}{f^2}$, and its point of application may be assumed to be $\frac{7}{30} f$ below the crown. Let $E = 2{,}000{,}000$, $t = 50°$, $c = 0.0000055$, or $E\,c\,t = 550$.

From a change of temperature, the bending moment is found to be, at the crown,

$$\frac{24\,I}{f^2} \times 550 \times \frac{7}{30} f = 3{,}080\,\frac{I}{f};$$

and at the abutment,

$$\frac{24\,I}{f^2} \times 550 \times \frac{23}{30} f = 10{,}120\,\frac{I}{f}$$

For a width of 1 ft. of the arch ring, $I = \frac{12\,d^3}{12}$, where d = the thickness at the crown, the moment at the crown $= 3{,}080\,\frac{d^3}{f}$, and the moment at the abutment $= 10{,}120\,\frac{d^3}{f}$.

If we consider that the moment of resistance of the crown section for a width of 12 ins. (neglecting the reinforcement) $= \frac{12 \times d^2}{6} = 2\,d^2$, and that the moment of resistance at the abutment, when the thickness is twice the crown section $= \frac{12 \times 4d^2}{6} = 8\,d^2$, we can obtain the stresses in the extreme fibers caused by the change of temperature by dividing the moments by the section moduli, the simple expressions, $1{,}540\,\frac{d}{f}$ for the crown section, and $1{,}265\,\frac{d}{f}$ at the abutment, whatever the span of the arch.

The writer will endeavor to compare the two bridges shown as far as the few measurements given will allow. He will assume that the thrust per linear foot may be found by the formula, $\frac{850 \times 128.5^2}{8 \times 25} = 70{,}000$ lbs. The live load of 100 lbs. per square foot distributed over a width of 29 ft. will amount to $\frac{100 \times 29}{17} = 170$ lbs. if distributed over the two arch rings 17 ft. wide; and the bending moment in a three-hinged arch at the quarter span

$$= \frac{170 \times 128.5^2}{64} = 44{,}000 \text{ ft.-lbs. per linear foot.}$$

The crown thickness of the arch is 22 ins. and assuming that the thickness at the quarter point is 33 ins., the stresses at the quarter point may be found as follows:

The compressive stress

$$= \frac{70{,}000}{33 \times 12} = 177 \text{ lbs. per square inch.}$$

The stresses in the extreme fibers from the bending moment

$$= \frac{44{,}000 \times 12}{12\,\frac{33 \times 33}{6}} = 242 \text{ lbs.,}$$

or a maximum compressive stress of 177 + 242 = 419 lbs., and a maximum tension of 242 — 177 = 65 lbs.

The average thickness of the arch is $22 + \frac{2}{3} \times 11 = 29.33$ ins.

In a fixed arch it will be necessary to assume a greater thickness of the arch at the crown, on account of the temperature stresses; therefore, the thickness at the crown will be assumed to be 28 ins. and that at the abutment, 56 ins. The maximum bending moment caused by a uniform load, w, per square foot at the crown is about $\frac{w \times l^2}{320}$, and the compressive stress from the thrust $= \frac{70{,}000}{12 \times 28} = 208$ lbs. per square inch; the stresses in the extreme fibers, from the uniform load, $\frac{170 \times 128.5^2}{320}$ divided by $\frac{12 \times 28^2}{6} = 56$ lbs. per square inch.

As shown before, the temperature stress in the extreme fibers $= 1{,}540\,\frac{d}{f} = 144$ lbs. per square inch, or a maximum compressive stress of 208 + 56 + 144 = 408 lbs., and a minimum compressive stress of 208 — 200 = 8 lbs. per square inch. At the abutment, the thrust is 70,000 × 1.28 = 89,500 lbs. and the compressive stress $= \frac{89{,}500}{12 \times 56} = 133$ lbs. The maximum bending moment from a uniform load

$$= \frac{170 \times 128.5^2}{64} = 44{,}000 \text{ ft.-lbs.}$$

The section modulus at the abutment $= \frac{12 \times 56^2}{6} = 6{,}280$, and the stresses in the extreme fibers $= \frac{44{,}000 \times 12}{6{,}280} = 84$ lbs.

The temperature stresses $= 1{,}265\,\frac{d}{f} = 118$ lbs., or the maximum compressive stress = 133 + 84 + 118 = 335 lbs. per square inch and the maximum tensile stress = 202 — 133 = 69 lbs. per square inch. The average thickness of the arch $= 28 + \frac{1}{3} \times 28 = 37.33$ ins.

The stresses from the shortening of the

arches will modify the results only very slightly in this case. Although both bridges have about the same stresses, the average quantity of concrete is only 27 per cent greater in the fixed than in the three-hinged arch. Or, assuming that 200 cu. yds. for the three-hinged arch is correct, the saving amounts to only 54 cu. yds., or $675, which is more than counterbalanced by the cost of steel hinges.

The authors state that the slightest movement of the abutments will develop cracks in the arches. This is not the case. A temperature drop of 50° is equivalent to a movement of both abutments of $\frac{1}{3,640}$ of the span, or, in this case to 0.42 ins., and it is a poorly designed abutment in which this movement will be seen. Such a movement would increase the stresses at the crown to 408 + 144 = 552 lbs. in compression and 144 — 8 = 136 lbs. in tension; and at the abutment to 335 + 118 = 453 lbs. in compression and 69 + 118 = 187 lbs. in tension. It is known that ordinary slabs do not crack under stresses less than 300 lbs. per square inch; hence the factor of safety against the first crack is still ample.

It is true, however, that the temperature stresses and those due to the shortening of the arch would be three or four times as great in the same arch with a rise of only one-tenth of the span, and the advantage of the three-hinged arch in such a case is unquestioned.

In a three-hinged arch, great stresses may be caused by the unequal settlement of the staging, and it has often happened that the hinges have been thrown out of line and out of center, and have had to be dug out and replaced. The failure of a large bridge was due to this cause. This is the main reason that fixed arches are preferred by experienced engineers to the three-hinged type.

It must also be mentioned that three-hinged arches settle considerably more than fixed arches after the centering is struck, and also cause more vibration under moving loads.

In three-hinged arches the use of deep ribs is a decided saving, which is not the case in all fixed arches. The writer has advocated the use of I-section for spans of more than 300 ft., and for spans of from 200 to 300 ft. he advocates T-sections.

The rule given by the authors for finding the center line for a three-hinged arch can be simplified by assuming the entire arch to be loaded with one-half of the uniform load per square foot. This, according to the elastic theory, gives the most favorable line of pressure.

Sewerage

Data and Discussion on Explosions in Sewers and Underground Conduits—Ordinances, Inspection, Ventilation and Other Preventive Measures.

The Department of Public Works of New Haven, Conn., warned by recurring explosions in the sewers and underground conduits of that city, and by reports of the same troubles in other cities, is still pursuing investigations begun more than a year ago to determine the causes of and to find means for preventing such explosions. The results of the studies made to date in this investigation are here given from information taken from a paper on the subject before the recent annual convention of the Connecticut Society of Civil Engineers by Mr. H. J. Kellogg, Assistant City Engineer of New Haven, who has charge of the investigation. Mr. Kellogg enumerates a number of instances where explosions in sewers and other underground conduits have occurred and several of these are here described on account of the present renewed interest in the subject due to recent explosions in sewers at Pittsburgh and Detroit.

The first explosions in New Haven which caused injury to persons occurred about 20 years ago in a 54-in. circular brick sewer when four men were inside the sewer cleaning it. The air in the sewer was burned out and it was with great difficulty that the youngest and strongest man in the group succeeded in getting his companions to a nearby manhole where they were helped out of the sewer by workmen on the outside. Two of the men were disabled for six months or over. In this case illuminating gas was present and was ignited by lanterns carried by the men. At that time, 20 years ago, the manufacture of by-products from gas plant wastes was very limited and some of these waste products got into the sewer. The gas plant was near the point where the explosion occurred. In those days gas drips were run directly into the sewer if it was handy to make connection. After that explosion greater care was used to ventilate the sewer by opening up two or more manholes before starting repairs and cleaning.

The next explosion in a New Haven sewer, which caused personal injury, occurred last winter in a 42-in. egg-shaped brick sewer when men were inside cleaning it of deposit. They were burned about their faces and hands. There was no evidence to show that this explosion was caused by the presence of illuminating gas. Near the place where the men were working, close to a manhole, there was a drainage connection from the factory of a paper box and paper concern. In this factory a benzine, naphtha or gasoline mixture is used for cleaning the stones upon which the lithographic work is done. Sometimes one of these liquids alone is used for the purpose named but not over a spongeful at a time. In this case positive evidence could not be secured of the accidental spilling of inflammable liquids. No garages were located above the point where the explosion occurred. The gasoline odor was noticed before the men entered the sewer, but manholes had been left open for a length of time that was considered sufficient to clear the sewer atmosphere of any dangerous taint. Lanterns of ordinary type were carried but will not be so employed again in New Haven. A small, neatly made, dry battery lantern, containing a small Mazda lamp, known as the "Ever Ready" lantern, is now used. Equipped with such a light, with smoking and the scratching of matches forbidden, there seems to be no chance of starting an explosion while cleaning or repairing the New Haven sewers.

The first New Haven sewer explosion occurred in 1886, when no one was inside and no one connected with sewerage work was in the vicinity. A 60-in. circular brick sewer was blown open, the arch was thrown completely off for about 100 ft. No one knows what combustible wrought the destruction, nor how it was ignited. It was the only explosion that ever harmed a New Haven sewer.

EXPLOSIONS IN CONDUITS OTHER THAN SEWERS.

A number of explosions have occurred in underground conduits for wires, some of them serious and expensive. In Bridgeport about two years ago a manhole cover was blown off by ignited illuminating gas and the gas kept on burning for hours before the unmetered supply could be stopped. The sheaths around the cables and the insulations were burned and the service was entirely suspended. Again in Bridgeport, about four years ago, an explosion in a conduit manhole broke 40 panes of glass in adjacent buildings. A riser pipe had been cut off just above the curb, and a ventilator cap furnished the little holes for illuminating gas escapement from the manhole and the cap happened to be right handy for a boy to strike a match on. In Fairfield a trolley car struck sparks from a gritty rail and touched off a blast that blew up 675 ft. of ducts. For an explosion in Hartford the gas company paid damages of $5,000 to the telephone company. Ventilated manhole covers have checked this line of conduit troubles, but a bad break in a gas main would threaten the underground systems as trolley car wheels and horseshoes strike sparks which might ignite the explosive mixture.

Columbus, Ohio, has experienced no explosions in sewers in 15 years. However, two explosions in underground ducts for telegraph and telephone wires have occurred in that city, caused by accumulation of natural gas. They were touched off by fire engines passing over the manholes and igniting the accumulated gas.

After the previously mentioned accident in the sewer last winter, we, in New Haven, felt the need of information from experiences of other cities: how they cleaned and repaired their larger sewers; what lighting apparatus was used for inside working, and how it was handled, so a round of visits was made to a few nearby towns, including New York (Borough of Manhattan), Providence, Boston and Worcester, and numerous letters of inquiry were sent out to city officials covering the points mentioned.

Albany, N. Y.; Newport, R. I.; New London, Conn.; Springfield, Mass.; Denver, Colo.; Portland, Ore.; Seattle, Wash., and Savannah, Ga., report no trouble from explosions.

Among cities that have suffered more or less from explosions, Louisville, Ky., may be mentioned. At Charleston, S. C., an employe lighted a cigarette and threw the match into a manhole just uncovered, and an explosion resulted. Also two men were caught by a flash in a sewer. At Syracuse, N. Y., two engineers were killed in 1911 by carbon dioxide. This has nothing to do with explosions, but is mentioned in passing. At Philadelphia one explosion recently, attributed to gasoline vapor, injured several men. The letter from Philadelphia was from George S. Webster, Chief Engineer, who, speaking of an incident pertinent to this discussion, said: "A suspicious smell was noticed upon opening a sewer manhole, and a blazing paper thrown in resulting in a column of flame about 40 ft. in height due to gasoline from a nearby factory." Two men were killed there about two years ago. This was caused by a leaky gas main that ran parallel with the sewer; and about 1½ years ago a man was burned from gasoline from a dry-cleaning shop. The sewers themselves were uninjured. Providence, R. I.; Buffalo, N. Y.; Washington, D. C.; Fall River, Mass.; Detroit, Mich.; Brooklyn, N. Y.; Cleveland, Ohio; Worcester, Mass.; St. Paul Minn.; Kansas City, Mo.; Duluth, Minn.; Los Angeles, Cal.; Savanah, Ga.; Baltimore, Md.; Hartford, Conn.; New Haven, Conn.; Pittsburgh, Pa., and San Francisco, Cal., also report explosions in sewers. The majority report illuminating gas as the explosive; in some cases known to be, others assumed; and where references were made to conduits for wires it was, in all but one case, so given.

In Baltimore men have been overcome by inhaling "sewer gas, notwithstanding the fact that every effort is made to ventilate and rid the manhole of such gases by use of a fan pump, as well as opening the manhole next to the one to be worked in, thus creating a current of air." The sewerage system of Baltimore was about as bad as could be found before the work of reconstruction was begun, and that is now well advanced. Probably many stagnant pools were formed due to faults in grades and deadly gases were generated. That city is now installing a comprehensive sewerage system, costing $20,000,000, thoroughly up to date, and it is difficult to imagine how any suffocating gases could be created in the new sewers.

George H. Norton, Deputy Engineer, Commissioner, Buffalo, N. Y., describes a new gas in sewers as follows:

> It may be interesting to know that for three years past at periods following thawing

> weather, after frost and heavy snow, we have had serious complaint from gas odor in some of our sewers. This has been traced down and found to be due to the use, by the railroads, of what is called a hydro-carbon oil which is used by them to prevent freezing of their switches and inter-locking plants in their passenger station yards, and at other congested points. An examination was made by the city chemist and he reports that at low temperatures this oil will crystallize, that such crystals are soluble in water, and for this reason he thinks that the odor or gases have passed the ordinary vent sewer trap. The gases arising from this oil are, apparently, highly explosive and could readily be the cause of a serious explosion. This oil is a by-product, from the Pintsch gas product used for car lighting. After a lengthy consultation with the railroad authorities, they have agreed to discontinue the use of this oil in their yards.

Roscoe N. Clark, City Engineer of Hartford, contributes the following information:

> I have no recollection of any explosions in the sewers previous to 1913. In 1913 we had two serious explosions in the Park river interceptor. The first, occurring in January, lifted one manhole head and threw several covers in the air, and also shattered windows in adjoining houses. The second occurred in May, when the East Side Pumping Station and its connection with the Park River intercepting sewer were in process of construction. The most damage was done to the station and adjoining houses. Both of the explosions took place in the vicinity of the works of the Hartford City Gas Light Co. Suits brought by property owners and the contractor for the pumping station are now pending in court. To protect the pumping station we are now building a ventilating stack connecting with the Park River interceptor and having a by pass equipped with an electric fan so tha if natural ventilation is not sufficient we can resort to artificial.

The correspondence here referred to, the chemical analyses that have been and are being made of sewage samples below garage centers and dry-cleaning works, and some inspection work to detect offenders, is being done to give New Haven a base upon which to build, if possible, an ordinance or set of ordinances that will help to keep sewers free from explosive gases. It is also hoped so to word those rules that no unnecessary burden will be laid upon an individual. Many cities have general clauses in their local laws that cover this subject that could be made to apply to continual and willful offenders. New Haven has a few rather poorly expressed paragraphs intended so to apply.

ORDINANCES.

Buffalo, Newark, Washington, Fall River, New Bedford, Philadelphia, Providence, Los Angeles and Savannah quoted their laws. The letter from A. E. Phillips, Superintendent Sewer Department of Washington, D. C., contained a clause fairly representative and a very good one it is:

> Section 18. No person shall make or maintain any connection with any public sewer or appurtenance thereof whereby there may be conveyed into the same any hot, suffocating, corrosive, inflammable or explosive liquid, gas, vapor, substance or material of any kind; and no person shall cause to enter or flow into any public sewer or appurtenance thereof any hot, corrosive, suffocating, inflammable or explosive liquid, gas, vapor, substance or material of any kind Provided, that the provision of this paragraph shall not apply to water from ordinary hot water boilers of residences.

Also, in Washington, there are rules for the installation of garage traps. We may not be able to improve on the ordinance quoted. In large cities it would be easier to compel the use of garage traps or oil separators than in smaller ones. That brings us, for the most conspicuous example, to the City of New York, Borough of Manhattan. Robert Adamson, Fire Commissioner, sent the following

> The inspector of combustibles of this department reports: Since June 5, 1912, there have been eleven sewer explosions for the reason that conditions in the sewers are so complex and opportunities for observation so limited that no definite knowledge can be formed of the conditions which lead up to and cause these explosions. There are no means to determine when an explosion is about to occur, and after such has occurred, no evidence is left to base an accurate opinion upon. It is only from the known conditions existing in the sewers that probable causes of explosions can be stated.
>
> The necessary conditions for evolving of gases, such as moisture, the presence of organic matter and bacterial activity resulting in decomposition, is always present, but just what part this action plays is absolutely impossible at present to state. The probable causes, at least in our experience in New York, are gasoline, illuminating gas and calcium carbide.
>
> Prior to the regulation requiring the installation of oil separators in garages, the larger number of sewer explosions was limited to what is known as the "garage zone" lying west of Broadway to the Hudson River between 40th and 60th Sts. Since their installation the number of explosions has been greatly reduced.
>
> The illuminating gas enters the sewer from ruptured, corroded or broken gas mains or from leaky joints, and when mixed in the proper proportions with air forms an explosive mixture. Calcium carbide which is used in garages for generating lights may be thrown into the sewer and when in contact with water generates acetylene gas, which is an intense explosive and is further auto-combustible, so that in many instances it may be ignited by its own heat generated in evolving the acetylene.
>
> The vapors of gasoline and illuminating gas require an open flame to ignite them, but how this occurs in sewers it is impossible to tell, but in vaults and in conduits the igniting spark is furnished in a number of cases by electrical apparatus, such as switchboards, sump pumps, short circuiting, etc.
>
> Some relation exists between sewer explosions and flooding of the sewers, and further when the sewers become tidebound these explosions are more frequent, such as following an unusual shower of heavy rain or a fall of snow or thawing of ice. This causes a condensation of the gases within the sewer.
>
> There are no casualties on record resulting from these explosions but considerable damage to property often occurs.
>
> In relation to places where oil is stored, the oil separator is the only method at present under the regulations of preventing volatile inflammable oil from discharging into the sewer.

From this and other reports and from exhaustive investigations and studies by the Bureau of Combustibles, of New York City, has come a set of rules and regulations that form a complete work on the subject; valuable to the authorities of any city as a book of reference. Even with this sizable pamphlet of local laws, to guide the city's officers and to warn offenders, entitled, "Regulations of the Municipal Explosives Commission," adopted Jan 3, 1912, the authorities of the city of New York are still having trouble in keeping their sewers in a safe condition. As has been stated there is a decided improvement, but stubborn opposition has been encountered A letter from the Ansonia Manufacturing Company of New York, which makes oil separators, says:

> The garage owners at one time combined and fought the ordinance in the courts, and after several appeals finally lost their case, with the result that this matter will be pushed harder than ever. Their contention was that it was an injustice to impose such an expense on the smaller garage owners and individuals, because they felt it was entirely out of their means to comply with this ruling For this reason it has been a difficult matter to sell the apparatus and afterwards to make collections for payment Some of the boroughs make a difference between private and public garages by calling any garage which stores more than two cars a public garage, and, under this classification, only public garages are subject to the ordinance.

That laws relating to certain uses or abuses that are made of sewers were not always fully enforced in the Borough of Manhattan the New Haven investigators had visible evidence. "No steam from an exhaust or direct blow off shall be allowed to enter the sewers." Yet, last winter, when we were investigating New York conditions, manholes were opened and dense columns of steam came out; and, testing by hand, it seemed very hot. With the steam came a pronounced odor of gasoline vapor.

Illuminating gases can be kept out of the sewers more easily than out of the conduits for wires, as the latter lie nearer the surface and afford better channels for the escapement of the gases that may accumulate under the nearly gas-tight pavements. In New Bedford it is intended to have the ducts caulked with oakum and some impervious substances like tallow that can easily be taken out and replaced. Also greater care is now generally taken in laying gas mains and caulking joints. But with gasoline it is different. We have found that in our own fire engine houses, now that auto trucks have come into use, careless disposal of small quantities of gasoline used in cleaning engines may have occurred, and, at the city's supply house, a like offense may have happened frequently. And from such smaller offenses we might continue a recital up to the greater ones committed, as we strongly suspect but have not yet proven, by the dry-cleaning shops. In the latter places white goods are first treated and then the liquid is used for goods that will not show a faint trace that would show on white fabrics and white gloves. When the oil gets too dirty it has to be disposed of, so I am informed. All of the napthas, especially gasoline, are common cleansers.

One method we are considering of caring for waste oil, either left in the vats of the cleaners or that has accumulated in small and not expensive garage sumps or catch basins, which we may insist shall be in every garage connected with a public sewer, is to require that the Department of Public Works shall be notified when the cleaners' vats are ready to be emptied or when the sumps or catch basins need cleaning; and then the department will send its detail to carry away the waste, clear the basins of oil and oily sludge, and dispose of this dangerous matter in a way to prevent explosions and fires.

We are not much impressed with this idea except for dry cleaning shops, and places where there may be an accumulation of waste inflammable oil. Garage sumps would not be likely to retain the oil as it would probably all evaporate. Oil separators have a receptacle to catch and hold all oils.

Naturally such a rule would apply to any considerable users of inflammable volatile oils No use of gasoline in factory, store or garage and no storage of anything above a trifling amount should be allowed except under a recorded permit for which a small fee should be paid. By regulations like the above, with clauses similar to the one quoted from the Washington laws, the dangers will be reduced if there is also careful inspection; even if the very efficient but expensive oil separators cannot be made compulsory. Constant vigilance is imperative no matter what regulations may be adopted or what apparatus is installed.

Very likely we shall always be threatened with the possibility of having illuminating gas get into sewers; but, as shortly before remarked, that danger is being reduced. For several years, in New Haven, we have not detected its presence, in any considerable quantity in our sewers

Chemical analysis of sewage to detect a dangerous amount of gasoline is almost futile. We have tried, without results, and may make further trials. The oil is so volatile that a film will not remain but a very short time unless there has been a large quantity run off; and the chance of getting a sample immediately after such a discharge would be a matter of luck. If volumes of the sewer atmosphere could be collected and tested there would be more likelihood of finding out if dangerous conditions existed. We may make the attempt.

If we could find out the cause or causes of ignition that have so far remained mysteries, we would be another step, and a long one, on our search for means of protecting ourselves against explosions in sewers. If the causes of the now mysterious explosions were known we might find simple coincidences in some while others might show complex combinations of coincidences.

One volume of gasoline oil produces 141 equal volumes of gas or vapor, as figured out by our chemist. Also our chemist gave as the computed best mixture for complete combustion: 1 part of the gas to 62½ parts of air. In gas engines, especially in automobile driving, an excess of air is said to give better practical results. I do not know how that would apply in sewer explosions. The following figures lead up to a knowledge of the amount of gas likely to be in a sewer from a given amount of oil:

1 gal. gasoline gives 141 gals. of gas.
141 gals. gas equals about 18.8 cu. ft. of gas.
18.8 cu. ft. of gas with 1,175 cu. ft. of air, best explosive mixture.

There is a broad variation in the proportions of this gas and air that will form combustible mixtures. Now allowing 8 cu. ft. as the available space for 1 lin. ft. of that 42-in. sewer, in New Haven, 1 gal. of gasoline could have furnished a prime mixture for about 150 lin. ft. of that sewer. A leaner but still dangerous mixture might more than double the danger zone. Although the rate of flow in the sewer, the time it would take to vaporize the oil, the draft in the sewer and perhaps other factors complicate the problem, the above figures show that a comparatively small quantity of gasoline is capable of making trouble.

Gasoline vapor is heavier than air. Illuminating gases are lighter. Therefore ventilating manhole covers furnish easy outlets for the latter; and the causes given of some of the explosions hereinbefore described (particularly those in telephone wire ducts) appear to be consistent. It might be well to state here that one part of illuminating gas to about seven parts of air is the computed best mixture for complete explosive combustion.

Constant ventilation to rid the sewers and conduits of these dangerous gases appears to be good practice especially to expel the illuminating gas. In Springfield, Mass., some of the catch basins are untrapped, and in Pittsburgh, one of the engineers spoke of trying that scheme. We saw perforated manhole covers in Pittsburgh. Some years ago, in New Haven, perforated manhole covers were tried. But the protests were so vigorous that the holes were plugged with bolts and now all covers are solid plated. There is ventilation through the manholes; but it is incidental to the frequent removal of covers, as there is always more or less work going that requires it, and also due to the fact that many covers do not fit tightly on the rabbets. Daily tides cause further ventilation. All outlets are below high water level. The objection against allowing sewer air to escape into the streets on the ground of the danger of inspection is without good cause. During the quarter of a century, or over, that Mr. Kellogg has been intimately connected with sewerage work, he has never heard of a sewer workman contracting typhoid fever or any other disease that could be laid to so-called "sewer air."

The Inspector of Combustibles of New York City, was quoted as saying that there appeared to be "some relations between sewer explosions and flooding of the sewers, and further when the sewers become tide bound. . . . . This causes a condensation of the gases within the sewers." If a sewer were bottle tight, except for its outlet, a high enough tide, accompanied by a heavy rainfall, would cause an internal pressure sufficient of itself to lift, and, in some cases, toss the manhole covers. But how that pressure could cause an explosion, granting, of course, the presence of a dangerous mixtures of gas and air, Mr. Kellogg could not find out. Some of the lighter air might be forced out if there were vents (a bottle tight sewer seems improbable) and a richer mixture formed with the heavier gasoline vapor coming from a coincidental injection of the oil. Then might come the mysterious ignitor.

A Brief Review of Recent Advances in Sewerage and Sewage Disposal.

During the past year there has been considerable growth of interest in problems of sewerage and sewage disposal throughout the country. This is shown by the increased work of the United States Public Health Service, which has undertaken extensive investigation of stream pollution, covering the water sheds of the Potomac and Ohio rivers and parts of the Mississippi River, as well as by a joint investigation of the International Joint Commission of Canada and the United States of the pollution of boundary waters. There has also been litigation of national importance to determine the rights of cities to dispose of their sewage severally and jointly in the waters of New York Bay, in which much interesting data have been collected, as yet unpublished. Another case of great importance to the sanitarian is the case of the Sanitary District of Chicago vs. the federal government, to determine the amount of water which may be abstracted from Lake Michigan, and the jurisdiction of the Secretary of War in limiting the flow. Within the various states substantial progress has been made in the state control of stream pollution through the growing regulation of sewerage and sewage disposal engineering.

With all these activities, a word of caution is not out of place. It is inevitable with the growth of cities that concentrations of sewage should result which must be discharged into streams or other bodies of water. It is not, as a rule, practicable to keep such water courses in their original condition absolutely free from pollution. Reasonable standards, therefore, of cleanness and freedom from nuisance must be adopted which will be shaped by local conditions, in particular by the surrounding circumstances and the use to which the streams and waters are put. In many cases, the economical and financial values need careful consideration, in order to escape enforcing expensive improvements to protect minor interests. Again, it should clearly be borne in mind that when the question of pure water is at stake that pure water can usually best be secured for drinking purposes by treatment of the water itself by filtration or other known methods, rather than by endeavoring to purify the sewage to a high degree before discharge into the drinking water supply.

The committee on sewerage and sewage disposal of the Illinois Society of Engineers and Surveyors devoted a portion of its report submitted at the recent annual convention of the society to a brief review of the recent improvements or advance in the art of sewerage and sewage disposal. This portion of the report, which is here published, was prepared by Mr. Langdon Pearse of the engineering department of the Sanitary District of Chicago. Such reviews are altogether commendable in that they enable the busy reader to check up on progress in the line which interests him with a minimum expenditure of time.

One of the most striking features of the advance in the last year or two has been the more accurate knowledge of the loadings and limitations of the methods of sewage disposal and the emphasis to be laid on proper care and operation of works. In particular, has stress been laid upon the prevention of the occurrence of odors in the processes attending sewage disposal. In long outfall sewers, considerable thought has been given to suggestions of aeration or the use of bleaching powder to prevent the flowing liquid from undergoing anaerobic decomposition before reaching the works for treatment. Again, in the construction of long intercepting sewers, considerable thought has been given to the preparation of sewage before it is intercepted, in order to relieve the sewer of the need of maintaining velocities always high enough to keep heavy particles moving. In one or two instances, this has pointed to the need of preliminary settling, to protect the working of an intercepting sewer.

From the standpoint of treatment and sludge disposal, marked attention has been given to the double-deck type of settling tank, as exemplified by the Imhoff tank, and numerous installations have been made in the United States under widely varying climatic conditions. Unfortunately, adequate data are not always kept on the behavior of these installations, but such evidence as is available indicates that, as a means of settling sewage and of digesting sludge to reduce its volume to a minimum, the tank has been a success. Attention has been called to the fact that the tank is not a machine for removing odors from overseptic sewage, nor will it produce drinking water. In this way, the development of the use of the double-deck tank has remained on a far safer, saner basis than was true ten or more years ago on the introduction of the septic tank, when extravagant claims were made by ill-advised promoters. The effect of these old statements has remained today, and in some cases has retarded the introduction of efficient modern devices.

In the double-deck installations a few variations have been tried in isolated instances covering connections between the settling and sludge digestion chamber. The value of this has not yet been tested by extended use. A few examples are prominent of the introduction of separate sludge digestion tanks in connection with the frequent removal of fresh sludge as exemplified in the plant under construction at Madison, Wis., described in ENGINEERING AND CONTRACTING of Aug. 20, 1913, but in general, the separate digestion of sludge has been considered only a means for improving the operation of old existing tanks, in order to prevent the development of intense anaerobic conditions in summer.

Considerable attention has also been given to the sprinkling filter. A few covered installations have been built in northern climates, and are reported to be working reasonably well, even in Canada. As yet, the principal means of distribution is by nozzles, although a few plants have been equiped with splash plates, fed by overhead troughs. One installation has been made at Springfield, Mo., of the revolving bucket type of distributor, drawn over rectangular beds by cables and fed by a side trough. Although this type of distributor is comon in England, it had not hitherto been tried in the United States on a life-size scale, although a somewhat similar installation was operated on an experimental scale for several years at the testing station of the Massachusetts Institute of Technology. For the protection of the nozzles in a sprinking filter plant, the need of preliminary treatment between the tanks and filters is not as yet definitely established. The results at Atlanta would indicate that for many months with efficient preliminary settling tanks, intermediate treatment may not be necessary. However, one designer still advocates a combination of contact bed and dosing tank between the settling tank and the sprinkling filter. On the value of this procedure, no definite data have been offered.

The use of intermittent sand filters and contact beds has continued on a conservative basis. Intermittent sand filters are still used where sand is cheap and a high degree of efficiency is desired. Contact beds are still used where it is desirable to keep the sewage from appearing at the surface.

For the disinfection of sewage, either raw or treated, chloride of lime or chlorine gas are available. However, unless the circumstances are exceptional, such as the pollution of a water supply or the handling of the effluent of a contagious hospital, the use of a disinfectant has not been generally followed. The process is expensive and requires care, making it more difficult to work on small installations. The use of a disinfectant, however, has been suggested and used along the sea coast on raw or settled sewage to protect oyster beds and also during the summer months to reduce the pollution of waters used in bathing immediately in the vicinity of sewer outfalls. Considerable study is being given to this question at Cleveland, Ohio, where a testing station has been in operation for over a year.

From the standpoint of the practitioner handling matters of sewerage and sewage disposal for the smaller towns, the advance in the art has been gradual but not great, nor does it appear that there will be in the immediate future any marked advance which will revolutionize methods now available for the treatment of sewage of towns with populations of 15,000 and under. The improvements being made and the investigations of special laboratory workers are directed to economies which can be carried out by large installations, and the research is more directed to find the limits of efficiency and the maximum rates possible. The small installation cannot afford the careful skilled continuous observation available for large installations. The small installation is frequently neglected, therefore, and is usually built in such a way that it will require a minimum of attention. The new developments along the line of sewage treatment have so far tended towards increasing the amount of operating care and skill and need of laboratory supervision.

From the laboratory standpoint, considerable advance has been made in knowledge of the limits of various chemical tests specified in the standard methods in vogue, usually those of the American Public Health Association. In particular, a large amount of study has been given to the putrescibility of various liquids and the best methods of determining the same in order to overcome the limitation of existing processes. In this connection, a method has been found which bids fair to prove entirely successful and by which it will be possible in all probability to determine easily the relative stability of a liquid, the amount of oxygen required to aid the self-purification of the liquid, and by the study of liquids from this standpoint to enable the engineer to plan improvements and study conditions far more satisfactorily than in the past.

The question of treatment of industrial wastes is also assuming importance not only in the eastern states, where manufacturing conditions are becoming congested from the standpoint of stream pollution, but also in the less congested districts of Illinois. Attention is being given to the handling of tannery wastes; Packingtown wastes, and the creamery wastes. Attention has been given in many localities to the wastes there current, and it is recognized that in many cases it is only a question of time before the problem of handling industrial wastes will become a pressing one; in others the problem must be solved now. On the question of expense, no precedents have been developed as to the method of division.

To the man interested in the construction of sewers, very little new is afforded. Staple materials are still in vogue. Sewers continue to be built of tile, concrete, brick, and reinforced concrete. Hollow segmental tile have been introduced, and have been used to some extent in certain communities. Attention, however, is being given in the construction of separate sewers in water-laden ground to the necessity of making tight joints, and the use of joint compounds is growing. Circular shaped sewers are still largely in vogue, though in some communities, for special purposes, egg-shaped sewers are still being built, even on a large scale; and for large sewers, horseshoe shapes with comparatively flat floors are becoming more common in this section.

In the design of sewerage systems, the need of keeping the sewage moving with adequate velocities is being recognized, with the result that sewage pumping stations are becoming more frequent. The need of keeping all material moving and preventing deposits in sewage pumping stations is also controlling the design.

The need for more accurate data on storm-water run-off and for more rational methods for computing the sizes of storm water sewers has been evident. Automatic rain gages and water-level gages have been developed to a satisfactory working basis. In connection with a weir built into the sewer, a water-level gage can be used to give a continuous record of the quantity of sewage. Such a record in conjunction with a continuous record of the rainfall will give a true basis for determining the run-off.

From the standpoint of the dry weather flow in sanitary sewers and the influence of ground water on sewer discharge, some data have been collected. However, the mass of information available could be wonderfuly increased if every member of this society who builds a sewer would gage the ground water flow before the connections were made. Such gagings, accompanied by a statement of the length of sewer and the sizes, would be worth a great deal. Gagings of dry weather flow and wet weather flow on sanitary sewers, as well as on combined sewers would also be of much value, and under ordinary circumstances can be obtained without much cost, if provision is made in the original construction for a weir at a suitable point.

From the standpoint of costs of construction and costs of operation and maintenance there is still a woeful lack of data and lack of interest on the part of those who should be most interested in the collection of the same. Data are frequently given in imperfect shape, or so distorted that its useful application is prevented. This committee would urge upon all members of the Illinois Society of Engineers the endeavor to gather careful cost data, even though brief, and in particular to gather data on the cost of operation and maintenance of sewers and sewage pumping stations, wherever it may be within their power to do so. For the collection of such data and its report, a most useful form has been devised by the Sanitary Section of the Boston Society of Civil Engineers, which the members of this society could well follow, not alone for their private information, but for the use of the society as a whole and the service of the sewerage and sewage disposal committee in making up its annual statement. The committee urged that every member of the society who is responsible for the sewers of a community should endeavor to have his community make up its annual report along the lines of this standard form, and should endeavor to have these data in hand early enough for the use of the committee of this society each year. If this can be done, much useful information will be obtained of service not alone to the members of this society, but to all engineers interested in like pursuits.

Methods Employed in Making Reinforced Concrete Sewer Pipe in the Open During Freezing Weather at Louisville, Ky.

Contributed by G. D. Crain, Jr., Louisville, Ky.

The methods which are being used by the Lock Joint Pipe Co. in Louisville, Ky., in manufacturing Meriwether system reinforced concrete pipe for sewer work, are somewhat interesting with respect to the precautions taken on account of freezing weather, such as that generally to be expected during January and February in that city.

The difficulty, not to say the danger, of handling concrete in cold weather, on account of the uncertainty attending the question of whether setting and curing are seriously interfered with by freezing, is well understood among engineers and contractors, although the extent to which work of this nature is handled during periods of low temperature indicates that the risk is not greatly feared, even in cases where a failure would almost certainly involve loss of life as well as of money.

In the Louisville case, however, the company and the city engineering department have co-operated to see that no trouble shall be experienced hereafter on account of imperfect setting of the concrete pipe which is to be used in several thousand feet of sewer, under a contract which is being handled by the Henry Bickel Co. of Louisville. The company is already at work on a section of the job, running to nearly 6,000 ft. of monolithic reinforced concrete main sewers, ranging in diameter from 6 ft. to 6 ins. to 10 ft., and, in the meantime, the Lock Joint Pipe Co. has been busy getting out pipe for the smaller sections of the sewer.

The pipe is to be used in sizes ranging from 24 ins. to 57 ins., vitrified pipe being specified for the smaller sizes. The company has already turned out a considerable quantity of the pipe, much of which was poured and had become well set during the mild weather of early December; and as to this, of course, there can be no question of protection from cold during manufacture.

As to the remainder of the pipe, however, and as to that made during the last week of December, there existed a necessity of providing carefully against trouble on account of low temperatures, and the company has accordingly made arrangements for that purpose which are satisfactory to the city engineering department, as represented by Roy E. Burks, who is in charge of sewer construction under City Engineer David Lyman.

It might be mentioned, incidentally, that the city was moved to greater care in this respect by reason of the fact that on one of the earliest jobs in the city in which this type of pipe was used cold weather was encountered during the manufacture of the pipe; and while there has been no trouble with the sewer, on inspection subsequently made cracks were discovered, which were attributed to the fact that severe weather had prevailed during the making of the pipe, and that the pipe had not been allowed to set long enough.

A period of 21 days after pouring has been fixed by the engineering department of the city as necessary to elapse before any of the pipe may be placed in the ground and backfilling done; and this is a minimum, moreover, which the engineers reserve the right to extend in case of extremely cold weather, if it seems necessary.

In view of the fact that extra precautions have been taken, however, and that pipe is being turned out well in advance of the progress of the work, Assistant City Engineer Roy Burks stated that he does not believe there will arise any question regarding the time limit on this job, as to the pipe, as it will have been completed in all cases, apparently, before the pipe are placed, and the methods used seem calculated to eliminate any probability of trouble.

The work is done outdoors at a temporary plant; the difficulty of securing a building susceptible of being heated and lending itself readily to satisfactory handling of the pipe caused this plan to be adopted. In even moderately cold weather, therefore, all of the materials, except the cement, going into the mixture—that is, aggregate, sand, water and reinforcing material, are heated.

Immediately after a batch of pipe is poured the lot is covered with a heavy tarpaulin, and live steam is introduced among the pipe in the forms by means of several lines of pipe, for 21 hours, thus guaranteeing that a reasonably high temperature, as compared with freezing, will be maintained during that period. At the end of that time setting has so far progressed that the forms can be removed and further curing can proceed, practically regardless of the weather, without the aid of heating.

This being the case, the pipe are left exposed at the company's plant for several days, being placed at leisure along the line of the sewer, where they will be reached in the due course of construction operations depending upon the progress which is made on the monolithic concrete section of the job. In the meantime, as it is not probable that continuous freezing weather will be experienced, curing can progress to a satisfactory degree; and as the early setting is made practically certain by the means described, no trouble is anticipated with the pipe, no matter how cold the weather may be during the pouring.

Notes on Construction Plant and Methods Employed on Large Sewers in Detroit, Mich.

Two large trunk sewers are now under construction in Detroit. These sewers are known locally as the Morrell Street and Mt. Elliott Avenue sewers. The former is about five miles in length and is now nearing completion. To date two contracts have been awarded on the latter sewer and it is to the construction plant and methods employed on this work that the present article pertains, the information given being based on a recent article in Steam Machinery.

The work now going forward on the Mt Elliott sewer is on Sections Nos. 1 and 2. Section No. 2 is a three-ring brick sewer 9 ft. in diameter, and will probably be constructed entirely in tunnel. Section No. 1 is the outlet section, 3,360 ft. in length, and comprises three different types of cross section as follows: 2,660 lin. ft. of 9-ft., 3-ring brick; 338 ft. of 4-barrel, 2-ring brick, each barrel being 5 ft. inside diameter, and 330 ft. of 4-barrel, the four barrels lying in a rectangular reinforced concrete section 24 ft. 8 ins. wide and 6 ft. 8 ins. deep, each barrel being lined with one ring of brick and 5 ft. inside diameter. The connection of the 9-ft. cylinder with the 4-barrel section is 30 ft. in length.

The principal reason for changing from the 9-ft. cylinder to the 4-barrel section was to reduce the vertical height and keep the lower end of the sewer under the grade line of the street. The outlet end, 333 ft. in length, extends into the river to the harbor line. This section rests on a piling foundation. The piling are driven with a scow driver and are cut off with a swing saw to sub-grade, which is about 3 ft. 6 ins. below the surface of the water. There are five rows of piling, 5 ft. 11 ins. center to center along the length of the sewer, and each row is capped with 12x12-in. timbers which are fastened to the piling with 1-in. drift bolts. On top of the caps there is laid a floor of timber 6 ins. thick, 12 ins. wide and 26 ft. long. These timbers are tongue and grooved and the concrete section rests on this flooring.

A row of 6x12-in. Wakefield sheeting is driven inside of the outside row of piling before the flooring is placed. This sheeting is for the purpose of holding the clay fill, which is made up to the level of the floor, and which gives additional stability to the structure to withstand the current and ice pressure which is likely to come against it.

Work on Section No. 1 was started at Shaft No. 1 and a drift was run each way from this shaft. At the present date 800 ft. of tunnel has been completed in the south heading and about 700 ft. in the north heading. The work is carried on in two shifts, the day shift mines in each heading, and the masons work on the night shift and brick in what has been driven during the day.

Shaft No. 1 is equipped with an automatic electric hoist and elevator. The excavated material is hoisted to a platform above the street level and dumped into wagons. Small industrial cars of about ½ cu. yd. capacity are used to handle all materials. Brick and sand are loaded on to cars which run on a track at the street level and are lowered into the tunnel on the elevator. The shaft is also equipped with an electric driven centrifugal pump manufactured by the Kerr Machinery Co. This pump is self-contained and is driven by a vertical motor which is direct connected with the pump. The motor and pump are encased in a frame and the entire unit is lowered into the sump. The motor is waterproof and is protected by an iron shield.

All of the 4-barrel work and about 600 ft. of the 9-ft. cylinder next adjoining will be built in open cut. Work was started at the river and at the present time most of the 4-barrel 2-ring work is in place and the bell mouth joining the two forms of cross-section has been built and work is being carried on on the 9-ft. cylinder section in open cut. A traveling derrick equipped with a 1 cu. yd. Owen clam shell bucket is used to excavate the trench. This derrick is of all steel construction and carries a 40-ft. boom. A standard Clyde hoist 8¼ ins. by 10 ins. 3-drum with swinger furnishes the power for the derrick. One of the niggerheads on the hoist has been removed and a jaw clutch placed on the extended shaft which operates a traveling mechanism. The engine is of the link reversing type and by reversing it the derrick travels either backward or forward under its own power. The derrick was designed and built by the C. O. Bartlett & Snow Co. There is also on the work a pile-driver equipped with a 6¼ in. by 8 in. link reversing Clyde engine which is to be used in driving the sheet piling.

Mr. Robert H. McCormick is chief engineer for the city of Detroit, and this improvement was designed and is being carried out under his supervision. Mr. William L. Day is engineer in charge of the field work.

The contract for Section No. 1 was let to the John T. Walbridge Engineering Co., and Mr. John E. Rowe is superintendent for the contractors.

Drainage and Irrigation

Methods and Cost of Making a Topographic Resurvey on the Truckee-Carson Project, Nevada.

Contributed by L. E. Gale, U. S. Reclamation Service, Lahontan, Nev.

A portion of the country lying north and west of Fallon being irrigable and water for irrigation being available upon the completion of the Lahontan Dam, it was found necessary to make a topographic resurvey before deciding on a system of irrigation. The country had previously been mapped on a scale of 4 ins. to the mile and 5 ft. contour interval by plane table parties in 1907. This scale and contour interval was inadequate in detail and the resurvey was made on a scale of 400 ft. to the inch with a 2-ft. contours. The parties, under the direction of Mr. R. W. Hawley, went into the field during the latter part of August, 1912, establishing the first camp about nine miles from Fallon, the base of supplies, and in the northeast corner of the area to be mapped.

TRIANGULATION.

The first work was that of establishing a base line and placing signals for the plane table parties. Two knolls or sand dunes about 7,500 ft apart were chosen for the ends of the base line as the intervening distance was a bare, hard Playa which would simplify measurements. Playas are a hard adobe formation, smooth as a floor, bare and practically level. They are usually in a pothole or depression and, in this particular section, are very numerous.

Bronze monuments were first set to mark the ends of the base and the azimuth of this line was determined by observations on Polaris at elongation Prior to measuring the distance, the theodolite was set over the north monument, sighted at the monument on the south and 3x3-in. pegs 18 ins. long were centered every 100 ft. on this line by a preliminary measurement and the tops driven flush with the ground Accurate levels were then taken on the tops of the pegs and in making the precise measurements the ends of the tape were held on these pegs, correction for inclination being calculated from the exact difference in elevation as determined by the levels.

In all but a few cases the tape was fully supported throughout, eliminating corrections for sag. The few times when the tape hung free was in the raise from the playa to the tops of the knolls. A Lufkin 100-ft. steel tape was used, cross-sectional area being .0035 in., which was frequently tested by comparison with a standardized tape of the same make and cross-section. The correct amount of tension was put on the tape by using a spring balance scale and at every measurement the temperature was taken from a centigrade thermometer attached to the tape by rubber bands and shielded by a piece of cardboard from the direct rays of the sun. Each tape length was marked by a small brass nail in the pegs previously set.

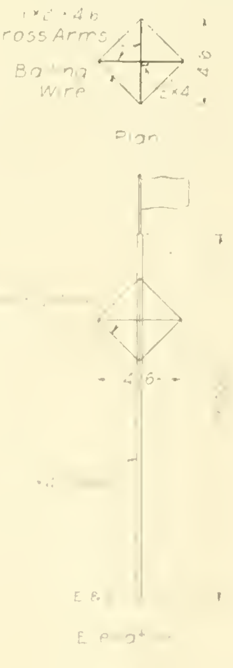


Fig. 1. Signal for Plain Table Work—Truckee-Carson Topographic Survey.

Two measurements of the base line were made, checking within .005 ft. after the corrections had been applied to each. This was due, possibly, more to luck than science.

The measured distance of 7215.911 ft. being adopted and reduced to sea level, the next step was to determine the latitude and longitude of each end.

The primary triangulation of the Coast and Geodetic Survey had been extended along the neighboring mountain ranges and cairns marking the triangulation points were easily discernable on clear days. The geodetic coordinates of these points were obtained and the position of the base line determined by the three-point method. A 10-in. Berger theodolite, with inverting telescope and double opposite verniers reading to 5 seconds, was used in all triangulation work.

Each new triangulation station was observed from two stations whose locations had been previously determined, the triangle being closed out at the new station. All angles were measured six times; three with telescope direct from right to left and three with telescope inverted from left to right, both *A* and *B* verniers being read each time. Form 1 shows the method used in recording observations.

The maximum allowable error of closure in any triangle was 5". The azimuth of the main scheme of triangulation was checked by frequent observations on Polaris, the tables of The Nautical Almanac being used in conjuction with this work. Whenever possible, the position of new points were determined from the base line.

The triangulation stations were located as near as possible on prominent points at the four corners of each sheet, giving the plane table men three signals to use in three-pointing and the fourth to check on. At each of these stations was erected a "diamond" signal 16 to 20 ft. high, the whole surmounted by combinations of colored flags for identification.

The triangulator and his assistant built signals and placed them in position over the triangulation points which were marked by 1¼-in. iron pipes 4 ft. long treated with two coats of tar paint, the caps being stamped with the proper signal letter or number.

Fig. 2. U. S. Coast and Geodetic Survey Cairn on Mountain Top.

After the necessary observations on each station were made, lattitude, longitude and azimuths were calculated from two known points, the work being kept in a computation book, and 15" co-ordinates plotted in ink on the plane table sheets by polyconic projection, using Wilson's Tables. The positions of the signals were plotted on the sheet by means of these co-ordinates.

Small flags, to be cut in by the plane table operators, were placed within the boundaries of a sheet when the character of the country necessitated more than the four signals on the boundaries. The signals were guyed by three wires to grain sacks full of sand which were buried in the ground. The bottom of the signal rested on the monument and was wired to a 4x4-in. post driven alongside.

In the beginning, the guy wires were merely fastened to the sage brush and the first windstorm blew all the signals down, making necessary long drives over the country to reset them One lesson of this kind was sufficient and thereafter the more secure anchorage was provided.

The diamonds were made by nailing crosspieces, 4 ft 6 ins. long, on 2x4s. Wires were strung from point to point of the crosspieces and from the points of the crosspieces to nails in the 2x4 placed 2 ft. 3 ins. above and below the intersection. The whole was then covered by white sheeting which was sewed to the wires Figure 1

LEVELS

Sea level datum was used in all the work, elevations being obtained from U S. G. S and U. S. R S. standard bench marks in the vicinity. The level circuits outlined the boundaries of each plane table sheet and bench marks, consisting of a wooden peg, guardstake, and small flag, were set every quarter mile. The work was carried to 100ths of a foot and the maximum allowable error in closure of any circuit was $.05\sqrt{D}$, where "D" equals the distance in miles around the circuit. The instrument used was a Gurley 22 in. Y-level. Near the close of the work, U. S. R. S. standard B. Ms. were set at or near all sheet corners and stamped with the adjusted elevation.

The triangulator and his assistant ran the levels and placed the permanent bench marks, the method of carrying out his work being to establish the triangulation stations, compute co-ordinates and plot them on the sheet and then run levels around the boundaries of the sheet, having everything in readiness for the plane table party.

Date—Nov. 3, 1912.
Weather—Clear, calm.

Angle at A.
Party—Gale-Snelling.

			DIFFERENCES.	
	Telescope Direct.			
	"A"	"B"		
C	60° 28′ 45″	240° 28′ 45″		
to			96° 33′ 35″	96° 33′ 40″
B	157° 02′ 20″	337° 02′ 25″		
	Telescope Inverted.			
	"B"	"A"		
B	202° 57′ 40°	22° 07′ 35″	96° 33′ 35″	96° 33′ 35″
to				
C	299° 31′ 15″	118° 41′ 10″		

Mean of seconds........36.25

$$\frac{96\ 33'\ 36.25}{3}$$

Mean angle...32° 11′ 12.083″

FORM 1.

PLANE TABLE DEVELOPMENT.

Each sheet, 24x31 ins., took in an area of 3 square miles; two miles E. and W. by 1½ miles N. and S., and the boundaries were made to conform with the section lines. A few years previous the section lines had been either retraced or resurveyed and practically all corners were found with little trouble. In some cases the corners had been covered by the shifting sand, but the majority of these were unearthed shortly after the approximate location was determined by projection on the map.

Two plane tables were operated, the party for each consisting of plane table man, recorder and two rodmen. The recorder entered the distances and rod readings in the notebook and calculated the elevations, calling the same to the instrumentman who plotted the position of each point and wrote the elevation by it. The two rodmen worked out the country in strips about 300 ft. wide, the width depending on the roughness of the country and varying in length or distance from the table with the visibility of the rod In this work the contours were not directly located, but a sort of cross-section of the country was taken, the rodmen giving the high and low points, general outlines of hills and pot holes, changes in slopes, low points in saddles and breaks in the contours in general. Each set-up of the table took in an area approximately 2,000 ft. square or 1,000 ft on each side of the table.

Fig. 3. A "Playa"—Level Plain of Hard Adobe.

After all shots necessary in each set up were taken, the plane table man walked over the ground and drew in the contours from the plotted elevations combined with personal observation; verifying doubtful contours by three-pointing at the spot and getting additional elevations where necessary.

The rodmen soon became proficient in their work, after a period of instruction, and when they understood just what was desired very excellent results were obtained. A great deal depended, of course, upon the topographical eye of the topographer.

The average area covered each day by each party was 0.3 square mile. The topographers set a standard for their work of 400 shots per diem and always tried to exceed this standard if possible; 400 shots in one day, in addition to sketching all contours in the field, locating section corners and moving between set ups, was usually a pretty fair day's work, but this mark was frequently exceeded. It can readily be seen that the number of

TABLE I—COST OF TOPOGRAPHIC RESURVEY FOR DISTRICT NO. 2, TRUCKEE-CARSON IRRIGATION PROJECT.

Classification.	Amount	Cost per sq. mile.
Horizontal Control—		
Labor	$ 294.86	$ 11.84
Corral expense	36.00	1.45
Supplies	25.98	1.04
Miscellaneous expense	94.90	3.81
Total for horizontal control	$ 451.74	$ 18.14
Vertical Control—		
Labor	$ 122.15	$ 4.90
Corral expense	18.50	.74
Supplies	9.77	.39
Miscellaneous expense	42.50	1.71
Total for vertical control	$ 192.92	$ 7.74
Plane Table Development—		
Labor	$1,294.77	$ 52.00
Corral expense	114.00	4.58
Supplies	19.14	.77
Miscellaneous expense	389.83	15.65
Total for plane table development	$1,817.74	$ 73.00
Draughting	162.05	6.50
General expense	252.71	10.15
Summary by Items:		
Labor	$1,711.78	$ 68.74
Corral expense	168.50	6.76
Supplies	54.89	2.20
Miscellaneous expense	527.23	21.18
Draughting	162.05	6.50
Total field cost	$2,624.45	$105.38
General expense	252.71	10.15
Totals	$2,877.16	$115.53

The item, miscellaneous expense, consists of idle time for men and teams, moving camp, equipment, depreciation and miscellaneous labor and supplies, which could not be charged directly to any of the classes of work shown on sheet. Location of work—Townships 19 and 20 N., R. 27 and 28 E., north of Carson River, in District 2 Area mapped, 24.9 square miles; rough, sandy country; scale, 1 in. equals 400 ft.; contour interval, 2 ft. Horizontal control developed from geodetic co-ordinates and maps projected from polyconic projection 41 linear miles of vertical control, 26 triangulation stations calculated and plotted, 15 permanent triangulation station marks placed 4 P. B. M.'s placed. Mess less of $90.79 is not included in the cost report. Average unit performance in square miles per plane table day 24.9, plane table days 94, unit performance 2 [illegible]

Fig. 4. Plain Table Party, Truckee-Carson Topographic Survey.

shots taken indicates much better than the area covered how hard the parties worked during the day.

The plane tables used were equipped with the "Johnson" movement and the alidades with "Beaman Stadia Arcs," both made by the W. & L. E. Gurley Co. While the stadia arc helps the recorder but little, it increases the efficiency of the table man to some extent and gives results that are easily checked mentally, thus decreasing the liability of error.

The stadia rods were made of 1x4-in. white pine boards 14 ft. long, the feet being painted solid black and white alternately and the fifth and thirteenth foot graduated to tenths.

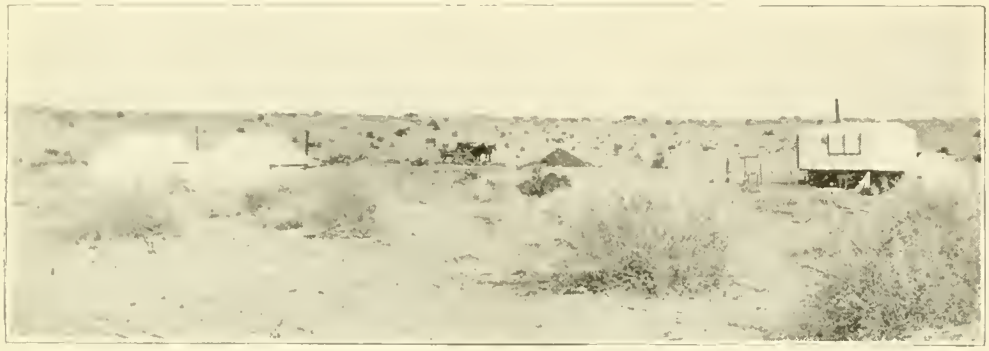


Fig. 5. Topographic Survey Party Camp. Truckee-Carson Irrigation Project.

The plane table sheets were "K. & E." No. 105 with cloth centers and cost approximately $1 each.

The parties were provided with two mule teams, one team being used to haul supplies and move the triangulator over the country; the other used by the party having the greatest distance to walk to and from work. The work was usually arranged to have one party working on the sheets nearest camp while the other party used the team and worked out the more distant sheets. Roads and trails were few and very poor, which necessitated much driving directly across the desert; in most cases no faster than a man could walk.

TABLE II. — COST OF TOPOGRAPHIC RESURVEY FOR DISTRICT NO. 3, TRUCKEE-CARSON IRRIGATION PROJECT.

Classification.	Amount.	Cost per sq. mile.
Horizontal Control—		
Labor	$ 285.51	$ 6.86
Corral expense	44.50	1.07
Supplies	23.53	.57
*Miscellaneous expense	114.00	2.74
Total for horizontal control	$ 467.54	$11.24
Vertical Control—		
Labor	$ 124.89	$ 3.00
Corral expense	28.00	.67
Supplies	12.52	.30
*Miscellaneous expense	53.98	1.30
Total for vertical control	$ 219.39	$ 5.27
Plane Table Development—		
Labor	$1,643.46	$39.51
Corral expense	65.00	1.56
Supplies	52.96	1.27
*Miscellaneous expense	556.00	13.61
Total for plane table development	$2,327.42	$55.95
Draughting	201.11	4.90
General expense	323.25	7.76
Summary by Items—		
Labor	$2,053.86	$49.37
Corral expense	137.50	3.30
Supplies	89.01	2.14
*Miscellaneous expense	733.98	17.65
Draughting	204.14	4.90
Total field cost	$3,218.49	$77.36
General expense	323.25	7.76
Total cost		$85.12

*The item "Miscellaneous expense" consists of idle time for men and teams, moving camp, equipment depreciation, and miscellaneous labor and supplies which could not be charged directly to any of the classes of work shown. General expense is administration Washington, D. C., Portland, etc. Corral expense is time of teams. Location of work: Townships 18 and 19 N., R. 27 and 28, south of Carson River, in District 3. Area mapped, 41.6 square miles; rough, sandy country; scale, 1 in. equals 400 ft.; contour interval, 2 ft. Horizontal control developed from geodetic co-ordinates, and maps projected from polyconic projection; 61 linear miles of vertical control; 36 triangulation stations calculated and plotted; 36 permanent triangulation station marks placed; 8 B. M.'s placed (bronze); 2 camps. Mess house loss of $106.66 is not included with cost report. Average unit performance in square miles per plane table day. 41.6 plane table days, 132 unit performance, .315.

Fortunately, water was obtained by boring wells at each camp which eliminated the heavy expense of hauling it. At the first camp, hot water was struck at 30 ft. which was slightly salt, but drinkable after being allowed to cool.

The weather was cool, as a rule getting colder, with snow, as the work progressed and many days were too windy for field work. These windy days the parties spent in camp, inking in elevations and contours on the sheets and checking calculations. The contours were inked with burnt sienna and elevations with india ink. At the close of the field work, two men were occupied one month doing this work and putting titles on the sheets.

The parties had breakfast at 6:30 a. m., left camp at 7 and took their lunch in the field, returning to camp at 5 p. m., the average time in the field running close to 8 working hours. No work was done on Sundays or holidays.

The area covered by the resurvey is divided into two districts; District 2 on the north side of the Carson River and District 3 on the south side. Two camps were made in each district and the work was started in District 2. This latter district was the most difficult of the two, particularly the northeastern part of it, which was very choppy and broken and had numerous playas surrounded by steep sand dunes, a few of the dunes being 40 to 50 ft. high and usually very irregular. The southeastern part of District 2, near Soda Lake, while rough also, was not as difficult, as the country was more regular and contained fewer playas.

District 3, while very rough in spots, was the easier of the two and the work in this district progressed rapidly. The northwestern part was very flat, contained few hills and the conditions for rapid work were ideal. The remainder of this district was not unlike District 2, the hills being about one-quarter of a mile apart, the ground descending sharply from the top of one hill only to rise abruptly to the top of another and, in some cases, the hills were 60 ft. high. A small area in the southwestern part of this district was the old bed of the Carson Lake and was very smooth and similar to the good country on the northwest.

The whole country in general was sandy, very irregular and covered with a dense growth of greasewood, sagebrush and other vegetation common to desert countries.

The weather was generally warm while working in District 2, and very cold at the close of the work in District 3, being below zero for over a week, with occasional flurries of snow.

The wages paid were as follows: Instrumentmen $100 per month, rodmen $60 to $70 per month, recorders $70 per month, teamster $60 per month, cook $60 per month. Each man was deducted 25 cts. per meal and the mess-house was intended to be self-supporting, including the wages of the cook. The mess-house was on wheels with walls and roof of heavy canvas stretched over a light wooden frame, the whole being easily moved from camp to camp by four mules.

Attached is a detailed cost summary showing cost per acre of the work in each district which includes the cost of moving the camps a total distance of 35 miles.

Cost of Irrigation Works With Special Reference to the Okanogan Project, U. S. Reclamation Service.

Sirs:—In Engineering and Contracting of Nov. 5, 1913, page 527, appears a comparative statement of the bid submitted and the actual cost of construction of the Conconully Dam, Okanogan Project.

I am much interested in this statement, as Mr. H. P. Holmes and myself were the bidders on this dam, and our bid was refused on the ground that it was too high. I am unable to locate a copy of the bid, but as we were the only bidders I presume that the figures shown in the statement cover our bid.

In looking over the statement, I am unable to see where any amount has been allowed for interest or depreciation of equipment.

I also notice that a charge against the bid under the item, "Difference in volume dam due to moving to new site, 177,000 cu. yds., at 59½ cts., $105,315.00." It will be noticed that the unit price used in the above mentioned item is the cost per cubic yard to the Government, while the bid under item, "Dam embk. exc. Cl. 2" was 55 cts. per cubic yard. It appears to me that an overcharge of $7,965 has been made against the bid.

I also notice that the General Expense, amounting to $18,000, has been estimated. This amount to me appears to be excessive, and I think it would have been a good idea to have shown a detailed statement of what items are considered in General Expense, and the amounts of each item.

I have been constantly seeing articles about public works officials who are making money with convict labor, but I have been unable to see where the Reclamation Service has made very much money by doing its own work.

With "Uncle Sam" as a banker it relieves the situation that confronts a great many contractors in all lines of work, as a solid financial backing is a wonderful help.

When officials on public work recommend that the work be done under their supervision that simply means that if they complete the work at a less cost than the lowest bid submitted, there has been a saving; if not, they put forth a fine explanation. My understanding is that no outside inspection is called in during the progress of the work, and probably none after the work is finished.

I am of the opinion that any contractor could have made a considerable saving under similar conditions, that is, to have his own inspection, keep the books, furnish his own distribution of costs, etc.

Mr. Holmes and myself also bid on the Cold Springs Dam, and were the lowest bidders. I have never seen the detailed cost of the construction of this project since it was finished, and if you have the data would be very glad to have you publish it along with the bid, as I am much interested to learn if the Government saved any money by doing the work under the direction of the Reclamation Engineers.

Very truly yours,
J. W. Sweeney.

Portland, Ore., Dec. 11, 1913.

(The foregoing letter has been brought to the attention of Mr. A. P. Davis, Chief Engineer of the U. S. Reclamation Service. His reply follows.—Editors.)

Sirs: Mr. Sweeney is correct in his statement that only one bid was received for the construction of the Conconully Dam, but the fifth annual report of the Reclamation Service, page 277, gives Mathew P. Zindorf of Twisp, Washington, as the bidder. There is no record of a bid having been received from Messrs. Sweeney and Holmes.

TABLE I.

Item—	Sweeney & Holmes lowest bid 6-28-06. Price.	Amount.	Lowest bids rec'd Sept. 17, 1906. Price.	Amount.	Actual cost. Price.	Amount.
xcavation for embankment:						
Earth, 158,000 cu. yds........	$ 0.31	$ 48,980.00	$ 0.29	$ 45,820.00	$ 0.28	$ 44,240.00
Gravel, 455,000 cu. yds.......	0.45	204,750.00	0.35	159,250.00	0.385	175,175.00
Excavation:						
Class 1, 61,300 cu. yds..........	0.40	24,520.00	0.40	24,520.00	0.295	18,060.99
Class 2, 19,300 cu. yds.............	0.60	11,580.00	0.55	10,615.00	1.37	26,441.00
Class 3, 3,100 cu. yds..............	1.25	3,875.00	1.25	3,875.00	2.84	8,804.99
Riprap (b), 80 cu. yds................	2.00	160.00	2.50	200.00	3.63	290.40
Rockfill (b), 28,500 cu. yds..........	1.50	42,750.00	1.75	49,875.00	1.46	41,610.00
Rockfill (c), 10,500 cu. yds.........	1.50	16,200.00	1.75	18,900.00	1.46	15,768.00
Drain pipe:						
6-in. laid dry, 1,900 lin. ft.........	.50	950.00	0.55	1,045.00	...	....
8-in. laid dry, 600 lin. ft...........	.60	360.00	0.75	450.00	...	....
8-in. laid cement jts., 600 lin. ft....	1.00	600.00	1.20	720.00	...	....
Concrete:						
Class A, 1,250 cu. yds...........	10.00	12,500.00	7.00	8,750.00	7.12	8,900.00
Class B, 500 cu. yds...........	11.00	5,500.00	8.00	4,000.00	7.12	3,560.00
Class C, 800 cu. yds..............	12.00	9,600.00	14.00	11,200.00	14.17	11,336.00
Class D, 560 cu. yds..........	14.00	7,840.00	11.00	6,160.00	11.09	6,210.40
	....	$390,165.00	....	$345,380.00	....	$360,395.70
educt bid on drain pipe*................	....	1,910.00	....	2,215.00	..	....
educt engineering and administrative expense included in unit cost of construction by government, but not included in bid prices......................	....	....	....	......	...	42,346.50
	....	$388,255.00	....	$343,165.00	...	$318,049.20

*There were no items constructed to compare directly with these items.

Mr. Sweeney states that he cannot see here any allowance has been made for interst or depreciation of equipment. By "interst" I assume that he refers to interest on unds invested by the contractor during the onstruction period. This is an item with which e Reclamation Service does not have to deal, ut, even if the corresponding charge that a ontractor must carry were made, this item ould not be large, as monthly payments are ade, and the full investment would have to e carried for only one month; the cost of is item would probably not be more than to 1 per cent of the total. Depreciation of uipment is fully covered in the actual unit sts of each item.

In regard to the addition of 177,000 cu. yds. $0.59½ to the contractors' bid, I may say that it would probably be more accurate to deduct this item from the actual cost of the dam as built by the Government; this would make the bid price and the actual cost $191,853.75 and $185,065.66 respectively, which makes the comparison more favorable for the actual cost than the original figures. In reference to the criticism of using the figure $0.59½ instead of $0.55, in comparing the cost of embankment, it should be noted that in the bid there was a classification of material excavated for embankment, the figures being $2.20 for class 1, and $0.55 for class 2. Both of these classes of excavation were included in the 177,000 cubic yards and in a comparison it is evident that it would not be fair to use either $0.55 or $2.20 and the actual cost for the entire embankment excavation of $0.59½ was therefore used.

In regard to the $18,000 allowed for general expense, this figure was used because the proportion of administrative and general expense of the actual building cost of the entire project to June 30, 1913, was almost exactly 11¼ per cent, which added to the estimated bid of $161,153.75 is over $18,000.

A rational comparison which would show the comparative figures in their true light is practically impossible because the location of the dam was changed, and it is not certain that the prices bid would have been the same on the new location.

Mr. Sweeney asks for data on the cost of the Cold Springs Dam, Umatilla Project. Bids on this work were opened June 28, 1906. All proposals were rejected and the work was re-advertised. The second set of proposals were opened on Sept. 17, 1906, and though somewhat lower than the first, they also were rejected. Table I shows the schedule of quantities and the lowest bid received under each call for bids; also for comparison the actual unit costs of the work as executed by the Government, including all general expense and overhead charges. It will be noted that a considerable saving was effected on the work covered by the bids. In the construction of the dam the quantities were increased somewhat and there were accessory structures not included in the work advertised which brought the total cost of the reservoir works up to $140,000.

If further details regarding this work and its cost are desired, they may be obtained from a paper by Mr. D. C. Henny on "Two Earth Dams of the U. S. Reclamation Service" in the Transactions of the American Society of Civil Engineers, vol. 74, December, 1911. An extract of considerable length from this paper appeared in Engineering & Contracting of May 24, 1911.

Very truly yours,

A. P. Davis, Chief Engineer.

Washington, D. C., Jan. 21, 1914.

Roads and Streets

nvestigation of the Causes of Expansion and Contraction of Concrete in Concrete Roads With Reference to the Prevention of Cracks.

Actual experimental data relating to the pansion and contraction of concrete in ncrete roads are given in a report to the ational Conference on Concrete Road uilding. These data were obtained through ecial studies by the Bureau of Standards d the report to the conference was pre red by Mr. R. J. Wig of the Bureau. The port in abstract follows:

ause and Effect of Expansion and Contraction of Concrete in Concrete Roads.

The effect of contraction may be finally idenced by cracking and that of expansion crushing, spalling or buckling. The en ncer is interested in this subject only so r as it affects the integrity of the road. e chief causes of expansion and contraon are: (1) Changes in the temperature of e concrete; (2) variation in the moisture ntent of concrete; (3) variation in the con ion and character of the sub-base; (4) ex ss loading by traffic.

F EFFECT ON EXPANSION AND CON[illegible] OF TEMPERATURE CHANGES IN THE [illegible]

It is generally considered that the variation the temperature from season to season tading to cause a change in length combined th unequal frictional resistance between the concrete and the sub-base, is the primary se of cracking. The change in the length concrete due to temperature [illegible] ove, as determined by Bouniceau, Christophe, A. R. Binnie, W. D. Pence, Professor llock, Professor Lyford [illegible] Professor Norton, ranged from .0000065 to .0000135 per degree F. per unit length, the accepted value being about .000006. Assuming a normal range of temperature of 90° F. the movement which would occur without restraint is about .00054 per unit of length.

In practical work, however, several elements enter tending to modify the temperature effect. A reduction in the moisture content of concrete would cause a contraction while an increase would cause expansion, thus aiding or counteracting the effect of temperature. The friction on the sub-base in the case of a road will always reduce the movement. In some recent experiments made by Henry S. Spackman upon experimental slabs of 1:2:3 concrete 18 ins. wide, 6 ft. 6 ins. long and 6 ins. thick, on a clay sub-soil a total movement of .00017 per unit length was observed under a change in temperature of 65° F. between July and November. The theoretical temperature movement for this range would be [illegible] per unit of length.

Measurements made by the Bureau of Standards on a concrete road on Morris Turnpike, near New Village, N. J., covering a period from October, 1912, to October, 1913, and in Nazareth, Pa., from June to October, 1913, show that the linear change in the concrete is not in accord with the temperature change. These measurements were made by stretching an invar tape, graduated to meters, along the road spanning several slabs and reading every second meter, the figures recorded as slab changes represent changes between the points nearest the ends of the slab. Readings were recorded to hundredths of a millimeter (.00039 inches). This is illustrated in Figs. 1 to 3, in which the linear changes occurring in a number of slabs have been plotted together with the mean temperature, moisture and weather curves. It will be noted that curve K, Fig. 3, shows a continual shortening of a 32-ft. slab. The concrete was placed June 9 and measurements started June 18. Between the latter date and Aug. 19 there is a considerable shortening of the slab, although the temperature had increased during this period. From Aug. 19 to Sept. 30 there was a drop in temperature and the shortening still continued.

The measurements plotted in curve Z-3, Fig. 2, were started five days after the concrete was laid. The curve indicates an elongation from November to April, when it reaches a maximum, followed by a shortening until August, when a decided shortening is evidenced and this is followed by an elongation until October. During this period there was a mean temperature range from 16 to 81° F.

In curve F, Figure 1, the linear change in a 50-ft. slab is plotted over the same period covered by curve Z-3, Fig. 2. Measurements were started on this course the day after the concrete was laid. During the winter months there is indicated an increase in length and from April until August a decrease. Curve F, Fig. 1, shows a small expansion and contraction during the first week but otherwise conforms to curve Z-3, Fig. 2. The changes plotted do not represent the differences observed with observations made upon the end reference plates only, but are obtained by summing up the changes observed at points two meters apart throughout the length of the slab.

The only explanation which can be given for the marked contraction in the slab plotted in curve K, Fig. 3, is that the slab was constructed on an old concrete road in which the cracks were sealed with cement mortar and ½-in. layer of clay placed upon this base to prevent a bond being formed between the new and old concrete. These conditions prevented the new concrete from absorbing moisture from below and permitted evaporation from the top, thus causing a shortening of the slab. This phenomenon has been demonstrated by laboratory tests.

The expansion indicated by curves E, Fig. 1, Z-3, Fig. 2, and K, Fig. 3 from November to April. could not be due to temperature influence and therefore must be due to increase in moisture content of the concrete. It will be noted from the precipitation curve that considerable rain fell during the latter part of March and the early part of April. The contraction from April to August is contrary to the condition which should result from a rise in temperature, and this can only be explained by a drying out of the concrete sufficient in amount not only to overcome the expansion due to a temperature increase of 20° F. but also to show a substantial contraction. From August to October the slab shows expansion which again can be explained by an increase in moisture content as there is no rise in temperature. It will be observed that this condition prevails in a number of other curves.

The only explanation which can be offered for an expansion similar to that indicated in curve E, Fig. 1, is the fact that the day after the concrete was laid the surface was thoroughly sprinkled with water and kept wet during the whole day, and there is also the possibility of temperature increase due to chemical action during the setting of the cement assisting expansion. The slab plotted in curve E, Fig. 1, together with all slabs of courses A, B, and C, were sprinkled more thoroughly than any of the other slabs.

There are no data available to indicate the expansion which might be expected in green concrete due to a rise in temperature brought about by chemical action during setting. Several of the curves of results obtained on courses A, B, and C, Morris Turnpike, would indicate that this setting heat might be effective in causing expansion. Temperature measurements have been taken in a number of cases, such as the Boonton dam in New Jersey, the Panama Canal locks, the Walnut Street bridge, Des Moines, Iowa, and the Keokuk dam, as well as in laboratories of Lehigh University and Watertown arsenal, and the temperature increases were noted to range from 17°F. to 108° F. in less than 18 hours after mixing. Such a great increase in temperature as these it is believed would have some effect, but its magnitude can only be determined by further investigation.

A variation in the quality of the concrete may also cause a variation in the thermal coefficient of expansion and it will of course affect the movement which takes place, as the modulus of elasticity of the concrete does change with a change in the quality.

THE EFFECT ON EXPANSION AND CONTRACTION OF VARIATION IN THE MOISTURE CONTENT OF CONCRETE.

It has been definitely established that with an increase in moisture content there is an expansion of the concrete and with a decrease in the moisture content a contraction. This phenomena is apparently true for all concretes at all ages. The magnitude of this change is not definitely known but experiments by Bauschinger, Schumann, Tomei, Considere, Campbell, White and Gary show:

(1) Neat cement hardening under water expands —0.15 per cent by volume, and the increased volume is approximately 0.08 per cent for a period of from 30 days to five years. The maximum expansion obtained at one year is nearly as great as at five years.

(2) Neat cement hardening in air contracts 0.25 per cent by volume in from 16 weeks to five years.

(3) Cement-sand mortars change in like manner but to a lesser degree.

Experiments by the University of Michigan show the following results:

(1) Neat cement hardening in air has an average unit linear contraction of —0.00109 at 7 days; —0.00190 at 28 days; —0.00236 at 6 months; —0.00270 at 1 year; —0.00289 at 2 years, —0.00322 at 4 years.

(2) That neat cement that has been hardened under water for three years will expand 0.0011 per unit of length and if then exposed to air for 60 days will contract 0.0005 per unit of length less than the initial length, and will therefore show a total linear contraction of 0.0016 per unit of length.

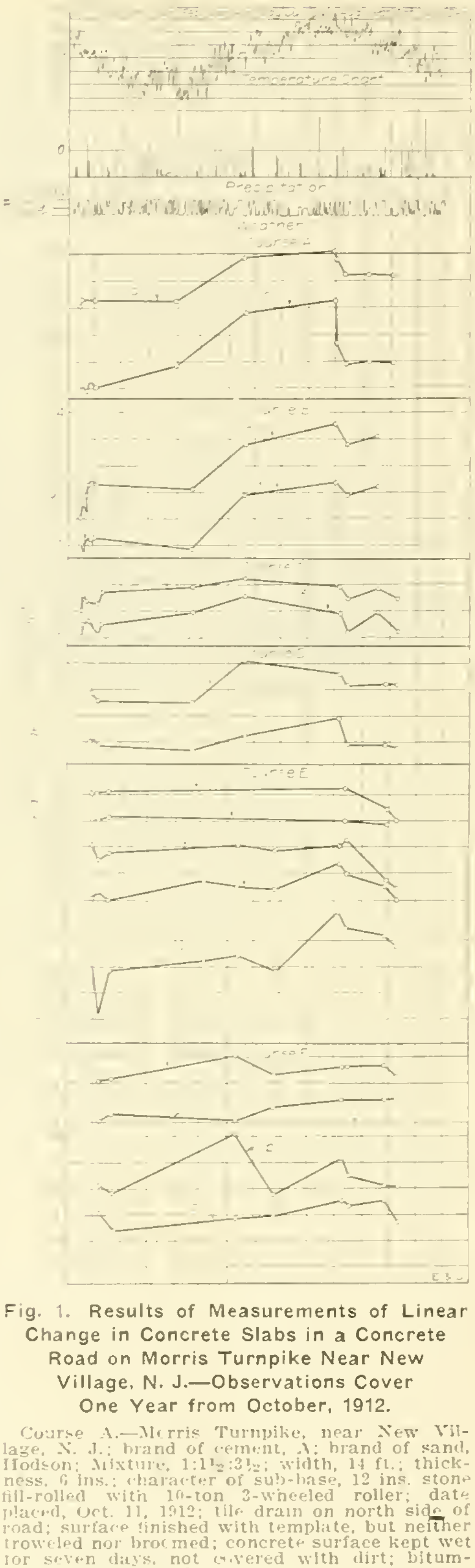


Fig. 1. Results of Measurements of Linear Change in Concrete Slabs in a Concrete Road on Morris Turnpike Near New Village, N. J.—Observations Cover One Year from October, 1912.

Course A.—Morris Turnpike, near New Village, N. J.; brand of cement, A; brand of sand, Hodson; Mixture, 1:1½:3½; width, 14 ft.; thickness, 6 ins.; character of sub-base, 12 ins. stone fill-rolled with 10-ton 3-wheeled roller; date placed, Oct. 11, 1912; tile drain on north side of road; surface finished with template, but neither troweled nor broomed; concrete surface kept wet for seven days, not covered with dirt; bituminous carpet covering.

Course B.—Morris Turnpike, near New Village, N. J.; brand of cement, B; brand of sand, Hodson; mixture, 1:1½:3½; width, 14 ft.; thickness, 6 ins.; character of sub-base, old macadam, scarified and reshaped, but old crown not entirely removed; sides brought up somewhat with new 1½-in. stone; tile drain on north side of road; date placed, Oct. 16, 1912; surface finished with template and wooden float and broomed; concrete surface kept wet for seven days, not covered with dirt; bituminous carpet covering.

Course C.—Morris Turnpike, near New Village N. J.; brands of cement, B & C; brand of sand, Hodson; mixture, 1:1½:3½; width, 14 ft.; thickness, 6 ins.; character of sub-base, old macadam, scarified and reshaped, but old crown not entirely removed; sides brought up somewhat with new 1½-in. stone; tile drain on north side of road; date placed, Oct. 16, 1912; surface finished with template and wooden float and broomed; concrete surface kept wet for several days, not covered with dirt; bituminous carpet covering.

Course D.—Morris Turnpike, near New Village, N. J.; brand of cement, B; brand of sand, Baker; mixture, 1:1½:3½; width, 14 ft.; thickness, 6 ins.; character of sub-base, old macadam, scarified and reshaped, but old crown not entirely removed; sides brought up somewhat with new 1½-in. stone; tile drain on north side of road; date placed, Oct. 18, 1912; surface finished with template and wooden float and broomed; concrete surface kept wet for seven days, not covered with dirt; bituminous carpet covering; short longitudinal crack started in slab of diagram g.

Course E.—Morris Turnpike, near New Village, N. J.; brand of cement, A; mixture, 1:2:4; width, 16 ft.; thickness, 6 ins.; character of sub-base, old macadam, scarified and reshaped, but old crown not entirely removed; sides brought up somewhat with new 1½-in. stone; tile drain on north side of road; date placed, Oct. 28, 1912; surface finished with template and wooden float not broomed; concrete surface neither covered nor wet; longitudinal cracks in slabs of diagrams i to k.

Course F.—Morris Turnpike, near New Village, N. J.; brand of cement, B; mixture, 1:2:4; width, 16 ft.; thickness, 6 ins.; character of sub-base, old macadam, scarified and reshaped, but old crown not entirely removed; sides brought up somewhat with new 1½-in. stone; stone drain on north side of road; date placed, Nov. 5, 1912; surface finished with template and wooden float; concrete surface covered with dirt and wet for seven days; longitudinal cracks in slabs of diagrams n and o.

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(3) That neat cements under water show a linear expansion of from 0.0007 to 0.0015 per unit of length at the end of one year, and after that a very slight additional expansion.

(4) That neat cement alternately exposed to air and water will show results if platted that will form a regular saw-tooth curve.

(5) 1.3 mortars show linear changes in the same direction as those of neat cement, but to a lesser degree. Submerged bars show a linear expansion in the first few weeks of as much as 0.0005 per unit of length, but decrease slightly after that, and then expand later to a length greater than the maximum expansion of 0.0005. Bars of 1:3 mortar in air shrink in length to an average of 0.0008 per unit of length within three months.

(6) Experiments with sections of a top coat of a cement walk which had been laid 20 years showed that it expanded when immersed 0.0005 per unit of length and contracted the same amount when again dried.

(7) Experiments with a section of stucco two years old, from a brick house, showed a linear expansion when immersed of 0.0008 per unit of length in four days, and a return to its original volume when dried.

(8) Experiments with a section of cement walk in which a sample of both the top course and base were bound together showed that upon immersion the base reached its maximum expansion in 15 minutes, while it took the top coat three days to reach the same expansion.

"This is interesting partly because of the evidence of alternate bending stress in the concrete due to the more rapid expansion of the lower layer and partly bcause of the ultimate agreement in expansion of the top and bottom portion." This cement sidewalk was in good condition after 20 years' service.

A. T. Goldbeck of the U. S. Office of Public Roads reports experiments on this subject as follows:

The specimens used were 8 ins. in cross-sections and 5 ft. long. They were molded as columns so that one end would be absolutely free to expand and contract, due to any cause. Different proportions by volume and different consistencies were used. The specimens of very dry consistency required hard tamping to consolidate the concrete in the mold, while those of wet consistency were made by simply puddling the mixture into place. The amount of water used for the dry mixtures was 8.5 per cent of the weight of the dry materials. For the wet mixtures, from 10 to 12 per cent was used. The materials entering the concrete mixtures were Portland cement, bank sand and ¾-in. crusher run gneiss.

The initial readings on all specimens were taken as soon as the concrete had hardened sufficiently to remove the molds.

The specimens which were allowed to dry out immediately after molding began to shrink almost from the day they were made, although the very wet mixtures for the first few days showed a tendency toward slight expansion, amounting to about .00005 in. in the 5-ft. length of specimen.

The rate of shrinkage of the dry mixtures is very slightly higher at earlier periods than that of the wet mixtures. There is, however, very little difference between the shrinkage of the different mixtures, except that the contraction of the wet specimens is delayed. The unit contraction of the oldest specimen of very dry mixture is 0.00052 at the age of three months, and the rate of contraction after this age is very small. The results further indicate that the approximate contraction of concrete of the proportions and consistencies ordinarily used is 0.0005 per unit of length at the age of three months.

The specimens which were kept moist showed expansion as long as they were wet. However, as soon as they dried out contraction began. The indications are that dry mixtures are more sensitive to expansion upon being moistened than are the wet mixtures. The amount of expansion is, however, quite small, reaching as a maximum approximately 0.0001 per unit of length. It will be seen that the rate of shrinkage of the moistened specimens, when allowed to dry out, approximates that of the air-cured specimens, so that one of the effects of moistening the concrete for the first few days is to delay the shrinkage. It is probable that ultimately the amount of contraction of both air and water-cured specimens will be the same. A specimen which was kept in a moistened condition continuously, although not expanding progressively, showed expansion up to the last measurement.

The results of experiments made by the Bureau of Standards are shown in Table I:

TABLE I.—RESULTS OF TESTS OF LINEAR EXPANSION OF NEAT CEMENT STORED IN VARIOUS MANNERS.

Test piece (1 in. by 1 in. by 13 ins.) neat cement prisms.

Number of cements.	Manner of storing.	Age of test piece.	Change in length per unit of original length.
6	In water	30 days	+0.00095
8	In water	30 to 60 days	0.00105
1	In water	120 days	+0.00113
20	In water	6 to 9 months	+0.00152
6	In air	30 days	0.00150
14	In air	30 to 60 days	0.00167
4	In air	120 days	0.00211
40	In air	6 to 9 months	−0.00285

It was noted that prisms which had been stored in air, when placed in water at practically any age, began expanding, the expansion proceeding in a manner similar to prisms placed directly in water after molding. Similarly, prisms stored in water contracted when removed and kept in air.

Note.—The original reading was taken on removal from damp closet 24 hours after molding.

Investigations have not been extensive enough to form final conclusions on the effect of moisture on expansion and contraction, but the following statements may be made which are in part applicable to concrete roads.

All of the tests quoted above show expansion of neat cement and of mortar and concrete when the samples are hardened under water. The results show the extent of the expansion to be greater with neat cement and to decrease when the addition [illegible] coarse aggregate is made.

These tests also show that neat cement, mortar and concrete will contract when hardened in air and that the contraction of neat cement is the greater, while that of [illegible] concrete varies with the [illegible] coarse aggregate used in the [illegible]

They show that to an [illegible] possibly for all time, the [illegible] looked for in concrete.

They show that the [illegible] provide for a decrease [illegible] when the temperature [illegible] increase in the moisture [illegible] perature decreases [illegible]

All these tests m[illegible] moisture content is very much greater than the effect of temperature change and may be sufficient to cause a stress in the concrete opposite to that which would be caused by a normal temperature change.

A variation in the quality of the concrete will cause a variation in the tendency to expand and contract with change in moisture content, as dense mixtures absorb water less rapidly than porous mixtures.

If concrete of two qualities in a road were exposed to an equal amount of moisture for a short period, the more porous of the two would have a tendency to expand the most and [illegible] strength [illegible] slight change in temperature and that the [illegible] resistance [illegible] uniform the more porous concrete would have to move the greater distance, higher tensile stresses would be set up upon drying out and it would be the first to crack.

If the concrete work [illegible] two quantities in the same slab of a road were similarly exposed, the same tendency would be exhibited and this may, in certain cases, be sufficient to cause a separation of the two concretes or may cause a number of small cracks.

If in two course work the base is of a more porous concrete than the top, the tendency would be to have unequal expansion and contraction in the slab, due to different moisture content in these mixtures causing the bottom to move more than the top or vice versa. Also if the concrete road is subject to a heavy rain for a considerable length of time the bottom will be exposed to moisture in the ground for a longer time than the surface and the surface will be more or less dried due to exposure to the sun and consequently the top would tend to contract while the bottom would still be expanding.

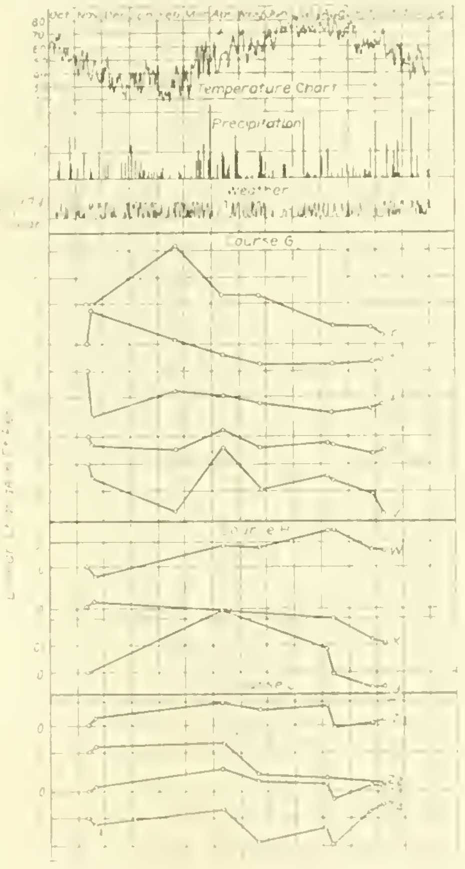


Fig. 2. Results of Measurements of Linear Changes in Concrete Slabs in a Concrete Road on Morris Turnpike Near New Village, N. J.—Observations Cover One Year from October, 1912.

Course G.—Morris Turnpike near New Village, N. J., brand of cement, A, mixture, 1-2-3; width, 16 ft.; thickness, 6 ins.; sub-base, old macadam scarified and reshaped, old crown not entirely removed, sides brought up somewhat with new 1½-in. stone, tile drain on north side of road; surface finished with template and wooden float, not broomed; surface of concrete covered with dirt but not wet; concrete placed Oct. 26, 1912.

Course H.—Morris Turnpike near New Village, N. J., brand of cement, B, mixture, 1-2-3; width, 16 ft.; thickness, 6 ins.; sub-base, old macadam scarified and reshaped, old crown not entirely removed, sides brought up somewhat with new [illegible] stone, stone drain on north side of road; concrete placed Nov. [illegible], 1912; surface finished with template and wooden float and broomed; surface of concrete covered with dirt and kept wet seven days.

Course I.—Morris Turnpike near New Village, N. J., brand of cement, B; [illegible] width, 16 ft.; thickness, 6 ins.; sub-base, old macadam scarified and reshaped, old crown not entirely removed, sides brought up somewhat with new [illegible] stone, stone drain on north side of road; concrete placed Nov. [illegible], 1912; surface finished with template and wooden float; surface of concrete covered with dirt and kept wet for seven days.

THE EFFECT OF EXPANSION AND CONTRACTION OF VARIATION IN THE CONDITION AND THE CHARACTER OF THE SUB-BASE.

The condition and character of the sub-base would affect the support of the road; the amount of moisture transmitted to and from the concrete; the action of frost; and the friction on the bottom of the slab. All of these factors affect the expansion and contraction of the concrete.

If the slab of concrete is not uniformly supported, sections may be stressed, causing tension or compression in an amount sufficient to exceed the strength of the concrete, resulting in cracking. A condition illustrating this point may be found where a concrete road is built over an old stone road-bed of less width than that of the concrete slab. The foundation at the sides, which is not so compact as that at the center, will settle more, causing expansion in the surface of the concrete slab possibly sufficient to cause a crack. While there is no experimental data available definitely establishing this point, it is believed the longitudinal cracks in Course "C," New Village, N. J., may be thus accounted for.

The amount of moisture transmitted to and from the concrete through the sub-base depends upon the porosity and density of the material in the sub-base. It is believed that this fact explains the difference in the action of slabs ploted in curves Fib. 3, K, and Fig. 2, Z-3, and referred to under "The Effect on Expansion and Contraction of Temperature Changes in the Concrete." The action of frost can only occur in a sub-base containing moisture in localities in which the temperature gets sufficiently low to freeze to a depth of the sub-base. This action is therefore dependent upon the porosity and capillarity of the sub-base material and the depth of the water table below the concrete. The effect of freezing and thawing is to place the slab in stress which may cause cracking.

From observations made it would appear that the cracks usually occur during a thaw rather than during a frost and this may be explained by the fact that the heat may be transmitted more rapidly through the shoulders than through the center of the road, causing uneven settlement. If the heaving action of frost does cause cracking it is usually due to raising the edges of the road, which will probably crack the lower side, which may not be apparent until the slab returns to its original position when the crack will appear on the upper surface.

The friction on the bottom of the slab is dependent upon the condition of the sub-base. The [illegible] movement [illegible] therefore the tendency [illegible] change due to change in moisture content and thermal change would be restrained. The [illegible] would [illegible] stresses [illegible] proportional to the [illegible] The [illegible] in the experiments [illegible] Referring to these [illegible] effect of [illegible] a theoretical [illegible] length [illegible] per unit [illegible] An inspection of the [illegible] recorded in Figures 1 to 3 also [illegible] movement were not in accordance with theoretical requirements. The change in moisture content is not known and [illegible] had something to do with the

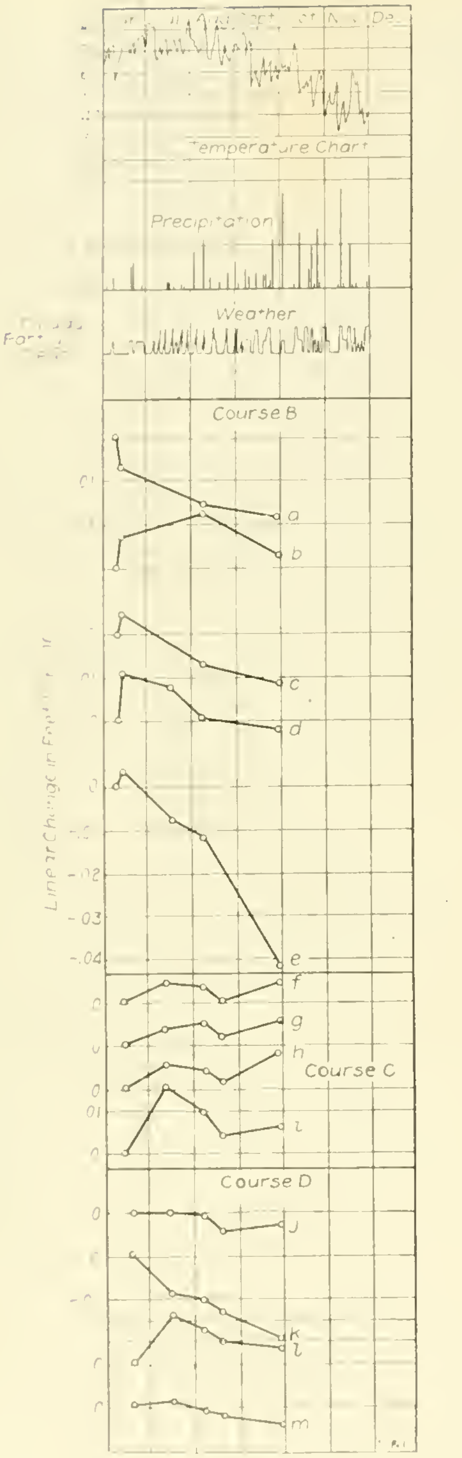


Fig. 3. Results of Measurements of Linear Changes in Concrete Slabs in a Concrete Road on Easton Road Near Nazareth, Pa.—Observations Cover Five Months from June, 1913.

Course B.—Easton Road, Nazareth, Pa.; brand of cement, D; kind of aggregate, limestone; mixture, 1:2:3, width, 16 ft., thickness, 7½ ins. at center, 5 ins. at sides; sub-base, 8 ins. Telford, taken up and replaced and rolled, and top sealed with loam. French drain on south side of road, concrete placed June 5, 6, 7, 1913, surface finished with wooden float and broomed crosswise, concrete surface sprinkled after hardening and covered with earth and kept wet for seven days.

Course C. Easton Road, Nazareth, Pa.; brand of cement, E; aggregate, limestone; mixture, 1:2:3, width, 16 ft.; thickness, 7½ ins. at center, 5 ins. at sides; sub-base, 8-in. Telford, brought to grade of sub-base, and sealed with loam, except Diagram i: tile drain on north side, French drain on south side; concrete placed June 9, 10, 11, 12, 1913; surface finished with wooden float and broomed crosswise; concrete surface sprinkled after hardening and covered with earth and kept wet for seven days.

Course D. Easton Road, Nazareth, Pa.; brand of cement, E; kind of aggregate, limestone; mixture, 1:2:3; width, 16 ft.; thickness, 7¼ ins. at center, 5 ins. at sides; sub-base, diagrams k, l, m, over old concrete base covered with loam; diagram j, 8 ins. Telford brought to grade and sealed with loam; tile drain on north side, French drain on south side of road; concrete placed June 9, 10, 11, 12, 1913; surface finished with wooden float and broomed crosswise; concrete surface sprinkled after hardening and covered with earth and kept wet for seven days.

discrepancies. Thus the actual movement observed by Mr. Spackman due to temperature may have been greater or less than that indicated by the figures.

The longitudinal grade of the road would affect the friction on the bottom of the slab. On a 5.2 per cent grade on Morris Turnpike a slab 30 feet long was observed to have moved down hill, the joint at the upper end of the slab being decidedly open while the lower end of the slab slid upward on the adjoining slab. The joint in this case was not perpendicular to the slab. Measurements also substantiate the above conditions. This sliding may have been assisted by changes in the thermal and moisture content of the concrete.

THE EFFECT OF TRAFFIC ON EXPANSION AND CONTRACTION.

The expansion and contraction of concrete is probably the least affected by loads brought upon the road by traffic than by any of the above mentioned causes. The effect is, however, not to be neglected, particularly when it acts in conjunction with an uneven settlement of the sub-base, similar to that heretofore mentioned of a slab being placed over an old stone road of less width than that of the concrete. The result would be cantilever or beam action and the consequent throwing into tension the top of the slab, which may cause cracking.

Application on Present Knowledge to the Prevention of Cracks in Concrete in Concrete Roads.

There are a great many factors affecting the expansion and contraction of concrete in concrete roads, and in order to present the available data in a clear and concise form your Committee has prepared a list of questions covering conditions which might arise in the mind of the engineer as influencing expansion and contraction. These questions have been answered in accordance with the available data.

Cracks.—Your Committee believes that with a proper understanding of the physical phenomena affecting the expansion and contraction of concrete in roadways, cracking can be eliminated.

(1) Longitudinal cracks usually do not occur until seasonal changes. Transverse cracks may occur at any time but the majority should occur during the first dry season.

(2) Transverse cracks are probably due to a favorable combination of moisture content in the concrete and atmospheric temperature conditions, together with a restraint induced by the condition of the sub-grade. They may, however, be caused by an unstable foundation.

(3) Longitudinal cracks are probably in the majority of cases caused by an unstable condition of the sub-base, although they may be caused by a favorable combination of moisture content in the concrete and atmospheric temperature conditions together with a restraint induced by the condition of the sub-grade. The latter is particularly effective if the sub-base is crowned.

(4) Diagonal cracks are probably most commonly caused by a combination of moisture content in the concrete and atmospheric temperature together with a restraint induced by the condition of the sub-base. In rare cases they may be caused by the above combination and longitudinal restraint along one side when abutting a rough structure.

Effect of Quality on Cracking.—(5) The best mixture is the one which is the most dense and which will reduce to a minimum the absorption of moisture, but it should also have a strength of at least 1,500 lbs. per square inch at the end of 28 days.

(6) Concrete should be mixed in such a way as to give a product of uniform quality.

(7) Concrete should be placed in such a manner as to give a uniform quality of concrete in place and to prevent loss of water.

(8) The sub-base should in all cases be thoroughly wet so as to prevent absorption of moisture from the concrete and thus cause a rapid drying out of the concrete.

(9) The concrete should be cured in such a way that it will retain its own moisture and receive sufficient additional moisture until it is strong enough to resist the shrinkage stresses induced by drying out.

Relation of Slab Length to Cracking.—(10) The maximum permissible length of slabs, of similar design, on a firm foundation, on a light or flat grade will vary with the climatic conditions. In localities where there is little rainfall, as in Arizona or the San Joaquin Valley in California, the slab length probably should not exceed 25 ft. This length has been determined by an analysis of a report by A. B. Fletcher, State Highway Engineer of California, on the cracking of a monolithic road, together with a knowledge of the results of measurements obtained by the Bureau of Standards. In localities where the rainfall is intermittent and the ground water rises in cool seasons and lowers in warm seasons, similar to conditions which obtain east of the Mississippi River, the slab may vary in length from 30 ft. to a continuous slab. While the 30-ft. slab may be regarded as a minimum length, from economical considerations, the results of experiments indicate that cracks are quite likely to occur under certain conditions even in this length. In many cases it will be observed that even in short slabs only slight motion has occurred at the ends, although there has been considerable motion within the slab, which has been absorbed at points of restraint. Other

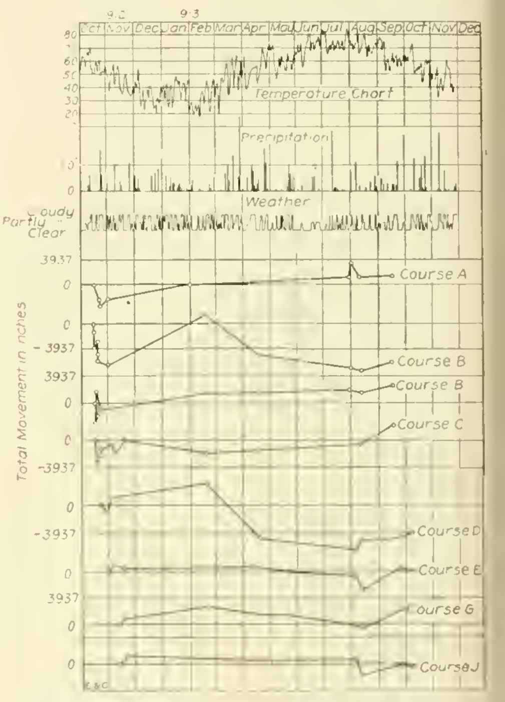


Fig. 4. Results of Measurements Across Various Types of Joints in Concrete Road on Morris Turnpike Near New Village, N. J.

One layer—two ply tar paper; Course A.—Between diagrams a and b; (slab a = 78 ft., b = 52 ft.).

One layer—two ply tar paper; Course B.—Between diagrams c and d; (slab c = 50 ft., d = 57 ft.).

Crack.—Course B; test of diagram e; (slab e = 50 ft., adjoining slab = 42 ft.).

Crack.—Course C; between diagrams e and f; (slab e = 44 ft., f = 70 ft.).

One-quarter in. wide; bituminous joint.—Course D; east of diagram h; (slab h = 49 ft., adjoining slab = 47 ft.).

Two layers 2 ply tar paper and one Baker plate.—Course E.—Between diagrams i and j; (slab i = 30 ft., slab j = 30 ft.).

Two layers 2 ply tar paper and 2 Baker plates. Course G.—Between diagrams u and v; (slab u = 25 ft., v = 25 ft.).

Two layers 2 ply tar paper and 2 Baker plates. Course J.—Between diagrams z_3 and z_4; (slab z_3 = 30 ft., z_4 = 30 ft.).

factors affecting the length of the slab are discussed in other questions.

(11) The data available to the Committee would indicate that the length of the slab for the same sub-soil conditions does not affect longitudinal cracking. It is, however, interesting to note that in a report by A. N. Johnson, State Highway Engineer of Illinois, is to be found a statement that in roads constructed by him in 1912 with a total length of 2.3 miles he finds 9.1 slabs per mile, cracked longitudinally, transverse joints being spaced from 50 to 100 ft. apart, on roads 16 to 18 ft. in width. A report of F. F. Rogers, State Highway Commissioner of Michigan, on the Wayne County Roads, shows that on roads constructed in 1912 with a total of 9.6 miles, 9.5 slabs per mile cracked longitudinally, transverse joints being spaced 25 ft. apart on roads 15 and 16 ft. in width.

(12) The length of slab should vary with the quality of the concrete. In a short time a denser concrete will absorb less moisture and consequently will have less tendency to move; it will also have greater strength to resist stresses induced by restraint of the sub-base. A dense concrete is particularly of value where subjected to intermittent wetting and drying. In a dry climate where plenty of water is not available for curing the slab length should not exceed 20 ft.

(13) The length of the slab should not be the same for all sub-soil conditions. If the sub-soil is of unstable material the length of the slab must be shorter than for a condition in which the sub-soil is stable.

(14) If there is a distinct variation along the length of the road in the materials of which the sub-base is constructed a joint should be placed where the change takes place. This is necessary because of the change in the frictional restraint offered by different materials and also because of the difference in the porosity which changes the moisture conditions.

(15) The length of the slab should vary with the condition of the sub-grade. If the sub-grade could be made very smooth so that a more or less uniform frictional resistance would be obtained throughout the length of the slab, it would perhaps be the most desirable condition, but as this is not obtainable it is believed that a more or less uniformly rough sub-grade is preferable to a so-called "smooth" sub-grade. This is explained by the fact that the rough sub-grade will make each small section in the slab care for its proportionate share of the stress induced by frictional restraint, while if the sub-grade is so-called "smooth" there will be rough places in it at intervals which will place upon certain sections undue stresses, which therefore will be more liable to cause cracks. If there is any decided change along the length of the road in the condition of smoothness of the sub-grade a joint should be placed at the point where this change occurs.

If the sub-grade is so called "smooth" on a steep grade, the permissible length of slab would not be as long as a light grade, there being a tendency to slide down hill and thus add additional stress to that imposed by moisture and temperate change.

(16) If the grade is steep the slab should be shorter in length than that used on a flat grade. A joint should be placed at all decided changes in grade.

(18) It would appear that the slabs could be made longer if constructed during the fall or winter, in localities where not subjected to freezing than if constructed in either the spring or summer, although there is not sufficient data available to determine whether this would permit of an appreciable change in length. Conditions indicate that concrete laid in the fall is probably not subjected to stress as much during the first few months since the tendency to increase in length due to moisture is counteracted by the tendency to decrease in length due to temperature.

(19) The experimental results indicate that there was greater movement of the concrete covered with a bituminous carpet than with concrete uncovered. This is indicated in curves shown in Fig. 1 [illegible] covered and uncovered slabs. If, upon further investigation, this condition is found to hold true the slab length would probably have to be varied. The reason for the greater movement noted is probably to be explained by the fact that a dark surface absorbs more sun heat and the bituminous coating holds for a longer period any moisture which gets beneath it.

(20) The length of the slab may be made greater for a greater thickness of slab. The increased friction caused by increased weight is proportionately less than the increased ability of the concrete to resist stress. Sufficient data are not available to give a relative figure.

(21) It is not believed that traffic need be considered in determining the length of the slab.

(22) There would tend to be a greater movement in a wide or elastic joint than in a narrow or unelastic one and since cracking occurs during contraction there would be less movement with a tight joint; a tight joint therefore would permit of a longer slab. It is not known, however, that this factor is sufficiently appreciable to be considered in determining the length of the slab.

Relation of Cross Section of Slab to Cracking. (23) A concrete road laid on a crowned sub-base would be the most liable to crack longitudinally and on a dished sub-base the least liable to crack. The resistance offered to contraction of a slab with a crowned sub-base is greater than that offered by either a dished or flat sub-base, because the concrete must move against the effect of gravity. With the dished sub-base the concrete contracts with the effect of gravity and therefore is stressed the least.

(24) The cross section having the greatest thickness at the center with a dished sub-base would offer the greatest resistance to longitudinal cracking; the cross section having the least thickness at the center with a crowned sub-base would offer the least resistance to longitudinal cracking.

(25) It is not believed that transverse cracks are in any way affected by the width of the slab. The data available to the committee would indicate that the width of the slab for the same sub-soil conditions does not affect transverse cracking.

(26) The slab should have its greatest thickness at the center no matter where the longitudinal drain is placed.

(27) Over an unstable sub-soil the slab should be made of greater thickness throughout the width than over a stable sub-soil, with the greatest thickness at the center.

(28) It is not necessary to vary the design of the cross section of a slab with the condition of the sub grade.

(29) The dished is the preferable design under all climatic conditions, especially in arid regions.

(30) The dished sub-base is the preferable cross section for all seasons and especially so if the concrete is laid during the dry season.

(31) The wider the road the thicker should be the slab.

(32) It is not necessary to vary the design of the cross section with the length of the slab.

(33) A concrete road which must withstand heavy and concentrated traffic must be made thicker than for light traffic.

The Value of Reinforcement in Preventing Cracking.—(34) Reinforcement will be of value (a) When the foundations are uncertain, (b) in regions where there is little rainfall and long or wide slabs are desired, (c) in regions where there is insufficient water for curing, (d) when it is necessary to have a thin slab, (e) where a decided change in grade occurs, (f) when the movement of the concrete is restrained at street intersections or on curves, if reinforcement is placed properly, (g) when the width of the slab is greater than 25 ft., (h) when it is necessary to crown the sub-grade and the slab can not be made thicker at the crown than at the sides.

(35) Only fabricated steel should be used and that form which will give the greatest distribution.

(36) The quantity of reinforcement can only be determined by an analysis of the conditions affecting each specific case.

(37) If uncertain sub-soil conditions tend to cause a settlement on the sides of the road, which is the most common condition, the reinforcement should be near the top. If settlement is apt to occur within the center of the slab the reinforcement should be placed near the bottom, and where the whole of the sub-base is uncertain the reinforcement should be placed near both top and bottom. Reinforcement to resist stresses due to temperature and moisture changes should be placed near the bottom of the slab. The stresses set up by a change in temperature are insignificant, except in arid regions, compared with those set up by moisture changes, and ordinarily reinforcement to provide for temperature stresses is unnecessary.

(38) The slab length may be anywhere from 20 ft. to continuous depending upon the many elements entering into the problem.

(39) The widest pavement without cracks within our knowledge is 40 feet. It is probable wider pavement can be constructed.

(40) It would appear that if properly designed a thinner slab may be used if reinforced. Definite information, however, is lacking to exactly determine this fact.

(41) The conditions of construction affecting cracking apply to plain concrete slabs as well as to reinforced slabs.

The Effect of Joints on Cracking.—(42) The purpose of joints is to relieve the stress which might be induced by a volumetric change in the concrete caused by change in moisture content or temperature and to care for unstable foundation conditions. The Committee has no evidence of the failure of slabs in concrete roads due to buckling, crushing or spalling and therefore consider the joint as primarily for the purpose of taking care of contraction.

(43) If a concrete road is constructed without considering the affecting factors, with the idea that nature will provide cracks, it will crack, but before doing so it will cause a weakening of the concrete for some distance on each side, which may result in a partial disintegration of the road. This is evidenced by short forked cracks appearing running off from the main crack. The cracks thus formed will gradually lengthen with time and will not only be irregular in alignment but they will not be vertical.

(44) The width of the joint controls the longitudinal movement of the slab. A greater width will allow of greater movement and consequently the slab will be more liable to crack upon contracting. See Fig. 1.

(45) A joint should be constructed as a contraction joint with little or no room provided for expansion, as all stresses in the concrete caused by expansion are compressive and may be absorbed by the concrete; later, upon contraction there will be little movement and therefore only small tensile stresses will be induced and there will be less liability of cracking.

(46) Experimental results show that only the movements occurring near the ends of the slab are usually transmitted to the joints.

(47) A filler should not be too elastic to comply with the conditions set forth [illegible].

(48) There are no available data to show the difference in effect of diagonal and square joints on expansion and contraction. Any joint, however, should be perpendicular so as to give proper bearing between adjoining slabs when they expand.

(49) It is necessary to provide a [illegible] joint between the pavement and an adjoining structure if the adjacent structure is rigid and will cause longitudinal restraint. If the adjoining structure will not offer sufficient resistance [illegible] being pushed out of alignment [illegible].

(50) Joints should be placed at street intersections and at sharp curves so as to take care of free movement, where [illegible] is [illegible].

The Effect of Character and Condition of Sub-base on Cracking.—(51) A rough sub-grade is preferable since a very smooth sub-grade is impracticable to construct.

(52) The sub-base should be very dense

but not entirely impervious. Where a rich mixture is used in the construction of a road it would tend to dry out unduly, causing excessive contraction. If a smooth sub-grade could be provided so that the friction would be reduced to a minimum it might be desirable to have an impervious sub-base, but this condition is very difficult to obtain.

(53) It is desirable to have the sub-base uniformly and thoroughly compacted so as to prevent uneven settlement.

(54) The sub-base should be dished.

(55) It would be desirable to have the sub-grade roughened on a steep longitudinal grade.

Methods and Cost of Constructing at Clinton, Iowa, Concrete Pavement with Bituminous Surface.

General methods and cost of constructing at Clinton, Ia., bituminous coated concrete pavement are given in a paper by J. G. Thorne, City Engineer, in a paper before the Iowa Engineering Society. An abstract of this paper follows:

The City Council decided to ask for bids upon two kinds of pavement, plain concrete and bituminous coated concrete. The contract for paving with bituminous coated concrete was let at $1.18 per square yard. Briefly the specifications were as follows:

After the proper subgrade was prepared and rolled a single course of Portland cement concrete base, uniformly 6 ins. in thickness, was placed. The cement used conformed to the standards of the American Society for Testing Materials. The mixture was composed of one part of Portland cement to two parts sand and four parts broken stone, or if gravel was to be used as the aggregate the mixture must be in the proportions of one part of cement and six parts of clean washed river gravel. Suitable expansion joints were provided every 25 ft. across the roadway. The transverse joints were not greater than ¾ in. in thickness and the longitudinal joints along each curb were 1 in. in thickness. These joints extended the entire depth of the pavement and were filled with asphalt to within ½ in. of the top. After the concrete had been laid as above specified and was perfectly clean and absolutely dry the bitumen was spread over the entire surface using not less than ⅓ nor more than ½ gal. to each square yard of paving. The bitumen was applied at a temperature of not less than 175° F. nor more than 225° F. In spreading this bitumen we found that a temperature of 225° F. gave the best satisfaction. Immediately following the spreading of the bitumen there was spread over the entire surface a uniform layer of dry, clean river gravel, graded by screenings through proper screens. Not less than 1 cu. yd. of gravel was used for 150 sq. yds. of surface. Enough of gravel was applied at all times so that the black surface of the bitumen was entirely covered. No bitumen was applied when the temperature was below 40° F. and the gravel was applied when the bitumen was sufficiently soft to unite with it. After the gravel was spread, the street was kept closed for a period of not less than two hours.

One of the chief difficulties encountered in the construction of this paving was the securing of clean gravel for the concrete base. Several barge loads were rejected at different times in order to secure the proper material. The gravel which contained clay or river silt would tend to destroy the bond between the top coat and the base, thus producing a failure. It was also difficult to get the concrete base absolutely dry and clean before the application of the bitumen and only the most rigid inspection secured this result. The concrete base was gone over with heavy street brooms two or three times and once with a house broom in the endeavor to remove all foreign substances which had collected. The gravel for the top coat was obtained from special beds near Bellevue, Iowa. The gravel was first screened at Bellevue by the dredging company, which took out all the larger sizes, and then screened again by the contractor, who used two screens set in the shape of the letter A. The first screen, ¾-in. mesh, was inclined 67½° with the pavement and the other, ½-in. mesh, was set vertical. By using these two screens the coarse and fine parts of the gravel were eliminated and the medium sized pebbles only were retaind.

The bituminous coated concrete paving in Clinton consists of 28,000 sq. yds. in two contracts, the price being as previously stated, $1.18 per square yard. I have no knowledge whatever as to the cost of the bitumen or the royalty paid. We have two blocks of plain concrete paving, one year old, consisting of 4,600 sq. yds. which were coated with Dolarway bitumen. The price, including the labor and all materials, was 15 cts. per square yard. The cost of the construction of this type of pavement varies considerably with the locality and prices of materials. The price of cement averaged $1.30 per barrel, net; gravel, $1.25 per cubic yard, delivered on the street; labor, $2.25 per ten-hour day, and teams $5 per ten-hour day. The wages in Clinton during the year 1913 were somewhat higher than usual, which may be attributed to the fact that there were four paving contracts aggregating 86,000 sq. yds. in addition to all the public and private work in the city.

As a basis of comparison I have taken two types of pavement—bituminous coated and vitrified brick. Owing to the inevitable tearing up of the streets we will assume that the life of any concrete base is only 40 years. The estimated life of a surface for bituminous coated concrete pavements and brick pavements has been limited to the period during which it is believed that the surface can be economically kept in repair. In comparing the annual cost of these two types of pavement it must be assumed that the kind or the type of the pavement is selected with due regard to traffic conditions. It is also assumed that ordinary repairs will be made promptly and efficiently so as to keep the surface in a satisfactory condition at all times.

The comparison, Table I, serves to show the relation in the annual cost per square yard between a low and a medium priced pavement.

TABLE I.

	Bituminous coated concrete paving.	Vitrified brick paving on concrete base.
First cost	$\frac{\$1.18}{40}$ $0.0295	$\frac{\$1.80}{40}$ $0.045
Life of base	40 yrs.	40 yrs.
Life of surface	5 yrs.	20 yrs.
Maintenance per sq. yd. per year	.01	......
Int. on original investment, 6 pct.	1.18×.06-.0708	1.80×.06-.108
Cost of renewing face	.15	1.25
Annuity required for replacement of surface, 4 pct.	.0276	.042
Average annual cost	.1084	.15

The above price of $1.18 for bituminous coated concrete and $1.80 per square yard for vitrified brick are the contract prices for paving in Clinton in 1913. The life of the surface has been assumed to be five years for bituminous coated concrete and twenty years for brick, at the end of which time the entire surface must be renewed. In view of the fact that the bituminous coated surface is to be renewed every five years, I believe that 1 ct. per square yard averaged over the whole contract is sufficient for ordinary repairs or patch work. Barring accidents, tearing up, etc., it is safe to say that the brick will not need repairs in twenty years. I know of certain streets in Clinton which were paved with brick on crushed rock base twenty years ago and with the exception of trench replacements the pavements have not received repairs of any kind. The cost of renewing the surface per square yard I have estimated for bituminous coated concrete at 15 cts. and for brick, $1.25.

Under the heading of annuity I have stated the sum that must be placed in a sinking fund each year, which with compound interest at 4 per ycent will equal $0.15 in five years for bituminous coated concrete and $1.25 in twenty years for brick surface.

The average annual cost, which is the sum of the maintenance, interest and replacements, shows $0.1084 for bituminous coated concrete and $0.15 for brick, a difference of $0.0416. In other words the property owners or city must pay .04 per square yard per year for the privilege of having a brick surface instead of bituminous coated concrete surface; or the brick pavement at $1.80 per square yard, which is equal in annual cost to that of the bituminous coated concrete paving, would have to last about 100 years without repairs of any kind. In comparing these types of paving I have also given the brick the benefit of the doubt in regard to the repairs as most authorities show that brick surfaces require some repairs.

By these comparisons I do not mean to infer that bituminous coated pavements are always more economical than the brick for all resident streets. I contend that the locality should always be taken into consideration before any definite statement can be made as to the relative efficiency of the different classes of paving.

The following advantages and disadvantages of bituminous coated concrete paving might be beneficial in deciding which type of pavement is suited to the street proposed to be improved.

Advantages of Bituminous Coated Concrete Paving.—(1) Low first cost. (2) Easily repaired at low cost. (3) Easily cleaned. (4) Comfortable to ride over. (5) Pleasing to the eye. (6) Does not absorb street liquids, is impervious. (7) Practically noiseless. (8) Produces no dust or mud. (9) Low tractive resistance. (10) Good footing for horses. (11) Does not reflect or radiate heat.

Disadvantages of Bituminous Coated Concrete Paving.—(1) Need of repairs at regular intervals. This probably is the most serious disadvantage of this type of pavement, as we are never sure that the succeeding city officials will understand or even care how the paving is being cared for. (2) The liability of being roughened by the stamping of horses standing in one place, especially when the paving is new. (3) Rigid inspection required in order to secure a pavement that will not be an absolute failure. This may not be classed as a disadvantage, nevertheness, it is a most important factor.

The bitumen used in coating concrete surfaces should be a tar from which the volatile portions below 200° C. have been almost completely driven off by heating, thus giving the substance the proper consistency. One of the most important constituents is the free carbon (matter insoluble in carbon disulphide) as that substance, in a tar, indicates the binding value. The bitumen may be obtained from the manufacture of either coke or illuminating gas. In making a set of specifications for this bitumen considerable latitude should be allowed as it is doubtful if different batches of exactly the same proportions can be made.

Production of Portland Cement in 1913.—Based on returns received by the U. S. Geological Survey up to Jan. 12, 1914, it is estimated by Ernest F. Burchard that the production of Portland cement in the United States in 1913 was approximately 92,406,000 bbls., as against 82,438,096 bbls. in 1912. The estimated shipments during 1913 were 88,853,000 bbls., as compared with 85,012,556 bbls. in 1912. Due to the large surplus of production over shipments the stock of cement at the mills apparently increased from 7,811,329 bbls. in 1912 to 11,375,000 bbls. at the close of 1913, or about 45 per cent. The relations between production and shipments for 1913 were the reverse of those for 1912, when shipments exceeded production. From the meager data at hand it is evident that the average price of cement per barrel in 1913 was appreciably higher than in 1912. Several plants located in the eastern and central states reported increases of from 10 to 25 cts. per barrel, although a few plants in the Rocky Mountain district reported slight decreases.

The National Conference of Weights and Measures, which is composed of state weights and measures inspectors, city inspectors, manufacturers of weighing and measuring apparatus, and federal officials, will meet in San Francisco during the Panama-Pacific exposition in 1915.

Engineering and Contracting

Devoted to the Economics of Civil Engineering Design
and to Methods and Cost of Construction

Volume XLI. CHICAGO, ILL., MARCH 4, 1914. Number 9.

Correction.

Our attention has been called to certain errors in the article on the proposed Rock Run storage dam for the additional water supply of Coatesville, Pa., which was published in the issue of Feb. 11, 1914. In Fig. 2 the force F' given as 39,375 lbs. is per foot of cut-off wall; per 15 ft. buttress it should be 590,000 lbs. This would increase the force R₄ to 1,952,000 lbs., the force H from 714,375 lbs. to 1,265,000 lbs., and the maximum pressure per square inch on the down-stream side of the buttress from 180 to 189 lbs. This would also decrease the pressure on the up-stream side of the buttress from 60 lbs. to 50 lbs per square inch. These errors existed in the matter from which the article was compiled and were overlooked by the editor.

A Novel Method of Indexing Our Articles.

Last autumn we devised and introduced a novel method of indexing all the articles that appear in this journal. If the reader will hold the back of this paper in his left hand, and bend the leaves with his right hand, he will disclose two black lines at the top edges of the leaves. Opening the paper at the first of these black lines he will find the index to advertisers. Opening the paper at the second of the black lines he will find the index to the articles in this issue.

A glance at this article index shows that every article is indexed under a general or class heading, for example, Roads. Then follows the title of the article and a condensed statement of its contents, the date of "E. & C.," and the number of columns occupied by the article. Not only does this form of index enable a busy reader to secure quickly an idea of the contents of all articles, but it lends itself to keeping an up-to-date card index of our articles. By clipping out the separate items in this index and pasting them on cards, the reader can readily make an excellent card index of all that appears in the columns of this journal.

In this connection we may well call attention again to the grouping of all our articles under section headings. This facilitates the examination of articles on a given subject, and it also makes it easy to bind—in loose leaf binders or otherwise—all the articles on a given subject. Thus, the reader who is particularly interested in roads, for example, can make an up-to-date book on roads, merely by placing in a loose-leaf binder the pages of our section on Roads and Streets. Bearing in mind that each of these pages contains as many words as are found in five pages of the standard 6x9-in. text book, it will be seen that our 28 pages of articles would fill a text book of 110 pages. Hence the contents of each of our sections for a year would make a very large book. A series of books thus compiled soon become invaluable particularly when a good card index enables the reader to locate any desired article.

Many, if not most, civil engineers and contractors are often obliged to work at a distance from their home offices. All these find it desirable to carry certain notes and books with them.

Readers who find it unnecessary to travel much may prefer to bind all their copies in one volume, semi-annually. And they may prefer to bind the pages of our weekly index, as well as our semi-annual index, in the same volume.

More than eight years have elapsed since this grouping of articles in sections was adopted by ENGINEERING AND CONTRACTING. Those of our readers who have the complete files for the last eight years will find an amount of usable information on any civil engineering or contracting subject that is surprising to any one who has not had occasion frequently to search through our files for all the data on a specified branch of engineering.

Electrolysis in Reinforced Concrete Buildings.

It is generally difficult to determine the direct cause of failure of reinforced concrete structures, and the facts are sometimes concealed or attempts are made to divert attention to other than the real causes of failure. There has been considerable discussion as to the possibility of damage to reinforced concrete buildings by stray currents from electric light and power sources, and in some cases the failure of reinforced concrete buildings has been attributed in part to electrolysis. Realizing the necessity of more authoritative data on the actual effect of electric currents on reinforced concrete, under various conditions, the United States Bureau of Standards has made some elaborate tests and field investigations, and has recently published a bulletin on the results of these researches. It was the purpose of the Bureau (a) to determine the nature and cause of the phenomena produced by the passage of electric currents, (b) to investigate the probable extent of danger from electrolysis in practice and the conditions under which trouble is likely to occur, and (c) to find means of mitigating the trouble due to electrolysis. On account of the thoroughness of the tests and investigations and the importance of a better understanding of the effect of electric currents on reinforced concrete we have included in our abstract of this bulletin the data and conclusions which are of most interest to engineers and builders. We have omitted the tabular data and that part of the discussion which is too technical in its nature to be of direct interest to our readers. The results of some of the tests and investigations are at variance with those of previous investigators, and on this account we have given a brief review of the conclusions drawn by previous experimenters.

It was found by this investigation that when an electric current of high voltage passed from the steel to the concrete the latter was cracked and broken, and that a mechanical pressure as great as 1,700 lbs. per square inch was produced under certain conditions, the rapid destruction being due mainly to the heating effect of the current. However, such high voltages as those used in the tests are not likely to occur in reinforced concrete buildings, and the danger from this cause is not great. An important finding was that the addition of a fraction of 1 per cent of salt to the concrete—salt being frequently added to prevent freezing of the concrete while setting—has a twofold effect; it greatly increases the initial conductivity of the concrete, thus permitting more current to pass through it, and also destroys the passive condition of the iron at ordinary temperatures, thus increasing several hundred times the rate of corrosion of the iron and the consequent tendency of the concrete to crack. It is therefore seen that salt should not be used in reinforced concrete structures which may be subjected to electrolytic action. Another fact determined was that the resistance of normal wet concrete to the passage of an electric current increases a hundred fold in the course of a few weeks, which greatly lessens the trouble from electrolysis.

Contrary to the observations of previous experimenters this series of tests shows that there is a distinct softening of the concrete around the reinforcement, which practically destroys the bond between the reinforcing material and the concrete, when the current flows from the concrete to the iron (cathode effect). This may prove serious in the case of reinforced concrete structures, for unlike the effect when the current flows from the iron to the concrete (anode effect), which only causes damage when the voltage is very high, the cathode effect is evident at all voltages, the rate being roughly proportional to the voltage of the current. As the softening cathode effect occurs only near the reinforcement, extending only a short distance from it in normal concrete, the crushing strength of the entire mass is not materially affected. The proposal which has been made to protect reinforced concrete buildings by maintaining the reinforcement cathode, either by using a battery or by other means, is therefore a dangerous one. To protect with the most certainty reinforced concrete buildings all electric power circuits within the building should be kept free from grounds. If possible, pipe lines entering the building should be provided with insulating joints outside of the building.

It was concluded from the experiments that concrete buildings without reinforcement are practically immune from electric currents, the only possible danger arising from the grounding of power wires, which then act as electrodes. It was also shown that it is much more difficult to apply a waterproofing which will prevent electrolysis than one which will insure a moderate dryness, and such agents, except waterproofing membranes, are ineffective; in fact even waterproofing membranes are of little value as ordinarily applied.

In making a diagnosis of a building to determine if there is danger of damage from electrolysis the fact that a fairly large voltage may be found near the structure should not be considered sufficient evidence that dangerous electrolytic conditions are prevalent. The distance between the reading points and the character of the intervening medium are of much greater importance than the mere magnitude of the voltage reading. As preventative measures, however, precautions should be taken to keep the potential readings as low as possible. The general conclusion drawn from the experiments and field investigations was that, under conditions such as are usually met in practice and which may give rise to damage from stray currents, the danger due to electrolysis has been greatly overestimated. Precautions are necessary, however, under certain conditions, but there is no cause for serious alarm.

The Washing of Concrete Aggregate.

We quote in full the following comment from the editorial section of the Concrete-Cement Age:

"There is no excuse for using dirty aggregate. From the use of a garden hose over wheel barrows with perforated bottoms, to the great power washing plants, there is material washing equipment to meet every need."

From the above we might infer that, by washing, any dirty aggregate can be made suitable for concrete. We entirely agree that there is no *reason* for using dirty aggregate, but we do not believe that it is always possible, from a commercial standpoint, to render dirty aggregate suitable for concrete, especially if the gravel or broken stone contains a considerable amount of clay. Engineers are being brought to a realization of the fact that it pays to inspect with care the concrete aggregate. It is, of course, necessary to use care in the selection of the cement and in the mixing and laying of the concrete, but attention should first be centered on the aggregate. Mechanical washing plants have an important field, but they can not be expected to produce a satisfactory concrete material from all classes of bank gravel and broken stone, at a cost which will not be prohibitive. Unless the engineer has a knowledge of the condition of the gravel or broken stone before washing he should exercise great care in inspecting the washed product before accepting it for use in concrete. The manufacturers of mechanical washing plants will do well to state the conditions under which their plants can not be expected to give satisfactory service, and to make some effort to investigate the conditions under which existing plants are working.

Buildings

Graphical Design of Eccentrically Loaded Columns.

Contributed by H. B. Cooley, Chicago, Ill.

The design of columns subject to cross-bending stresses in addition to axial loads involves a preliminary approximation, either as to size of section or unit stress, followed by investigation using a rather complicated formula. In some cases this process must be repeated several times before a suitable section is obtained, and it is to avoid this labor that the diagram of Fig. 1 was plotted.

Although the method used in constructing the diagram is general, the following limitations are involved for the particular graph shown in Fig. 1:

(a) The diagram as plotted considers only the I-section type of column, i. e., a column composed of a web plate and four angles, either with or without cover plates. The nominal width shown in Fig. 1 is that of the web, but the width used in plotting was the width of the web plate plus ½-in.

(b) The safe loads as given are for the strength of a column about an axis perpendicular to the web.

(c) The allowable unit stress is based upon the formula $16{,}000-70\frac{l}{r}$, where l = length of column, in inches, and r = radius of gyration about an axis perpendicular to the web.

The method of deriving the equations used in plotting the various lines of the diagram is herewith given, the nomenclature being as shown below:

S = unit stress in pounds per square inch,
l = length of column in inches,
L = length of column in feet,
r = radius of gyration about an axis perpendicular to the web,
P = axial load on column in thousands of pounds,
W = eccentric load on column in thousands of pounds,
h = width of column web plus ½ in.,
c = distance from neutral axis to extreme fiber,
a = area of column in square inches,
I = moment of inertia about an axis perpendicular to the web,
e = eccentricity of load, W, in inches.

Then

$$S=16{,}000-70\frac{l}{r}; \qquad (1)$$

also

$$S=\frac{P}{A}+\frac{Mc}{I} \qquad (2)$$

But $I=ar^2$ (3)

Substituting $r=0.4\,h$ in equation (3) (this value of r is approximate, but for sections in common use gives close results), we have

$$I=0.16\,a\,h^2 \qquad (4)$$

Substituting in equation (1) for r its value $0.4\,h$, and for l its value in feet; also substituting in equation (2) the value of I given in equation (4) we have

$$S=16{,}000-2{,}100\frac{L}{h}, \qquad (1, a)$$

$$S=\frac{P}{a}+\frac{M}{0.32\,a\,h} \qquad (2, a)$$

Equating values of S given in equations (1, a) and (2, a), we have:

$$16{,}000-\frac{2{,}100\,L}{h}=\frac{P}{a}+\frac{M}{0.32\,a\,h}. \qquad (5)$$

The above equation now contains five variables, and can be plotted in four quadrants, as shown in Fig. 1.

There are three general cases of column loading that may arise, any one of which can be solved by use of the diagram, as will now be shown by the solution of particular examples.

Case I: Axial Load Only.—In this case the last term on the right-hand side of equation (5) is zero; therefore only two quadrants are required for the solution.

Example: Column 15 ft. long, with 10-in. web plate; load = 300,000 lbs.; find required area of the column, for strength about axis perpendicular to the web.

Under the line marked "Length of Column in Feet" proceed vertically upward to the diagonal line marked 10 ins.; thence horizontally to the left, noting where the horizontal line intersects a vertical line passing through the given load 300 (thousand), which is marked on line "Axial Load P in 1,000 Lbs."; the required area is found to be 23 sq. ins. From the handbooks or from specially prepared tables this area can be made up to suit conditions. For example, four 6x4x½-in. angles and one 10x⅜-in. web plate give an area of 22.75 sq. ins., and r = 4.20 ins. The efficiency of this section is about 99.5 per cent.

Case II: Eccentric Load Only.—Replacing the eccentric load by an axial load and a couple equal to $W\,e$, we have:

$$16{,}000-2{,}100\frac{L}{h}=\frac{W}{a}+\frac{We}{0.32\,a\,h}=\frac{W}{a}\left[1+\frac{e}{0.32\,h}\right] \qquad (6)$$

Knowing any four of the above quantities, the fifth can be found directly from the diagram.

Example: Column 20 ft. long, with 12-in. web; load = 75,000 lbs.; eccentricity = 20 ins.; find required area of column.

Starting at line marked "Length of Columns in Feet," read around the diagram in a clockwise direction, considering the width of column and the eccentric load, in the second and third quadrants; then starting with line marked "Values of Eccentricity e, in Inches," read around the diagram in a counter-clockwise direction, considering the width of column; from the intersections in the third and first quadrants, project lines to meet in the fourth quadrant; their intersection gives the required area 35.6 sq. ins. A section can now be chosen which will give this area.

Case III: Combined Axial and Eccentric Load.—The allowable stress,

$$S=16{,}000-2{,}100\frac{L}{h}=\frac{P}{a}+\frac{W}{a}+\frac{We}{0.32\,a\,h}. \qquad (7)$$

Let $W\,e=(P+W)\,e^1$;

then $e=\dfrac{(P+W)\,e^1}{W}$.

Substituting this value of e in equation (7), we have:

$$16{,}000-2{,}100\frac{L}{h}=\frac{P+W}{a}\left[1+\frac{e^1}{0.32\,h}\right] \qquad (8)$$

It is seen from equation (8) that, for an application of Case III to the diagram, it is only necessary to use a value for the eccentricity, e, (shown on the diagram) equal to the actual eccentricity multiplied by $\frac{W}{P+W}$.

Example: Column 18 ft. long, with 18-in. web; axial load P = 120,000 lbs.; eccentric load W = 80,000 lbs.; eccentricity, e, of load W = 35 ins.; find required area of column.

First compute the value of e^1.

$$e^1=\frac{80{,}000\times 35}{200{,}000}=14 \text{ ins.}$$

Starting with this value of e^1 at the line marked "Values of Eccentricity, e, in Inches," and proceeding as in Case II, we find the required area to be 49 sq. ins. A section can now be chosen which will give this area.

In all cases care must be taken to see that the column selected is sufficiently strong about an axis parallel to the web. This can be done, either by making an investigation in the usual manner, or by plotting additional diagrams similar to those shown in Fig. 1, using the approximate values for the radius of gyration about an axis parallel to the web.

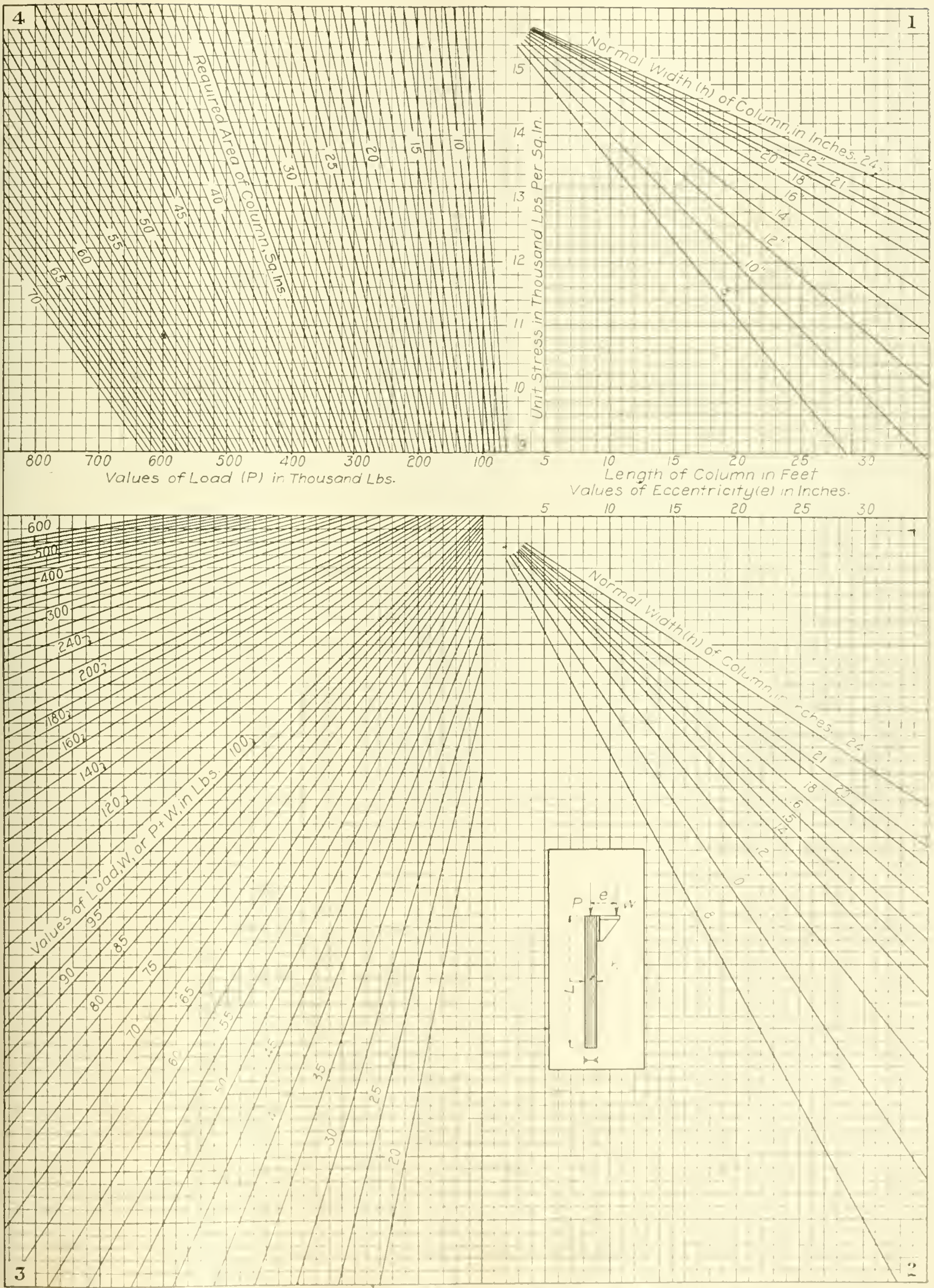
Summary of Test on a Reinforced Concrete Factory Building at Niagara Falls, N. Y.

An interesting and instructive test has recently been made on a new factory building of the Shredded Wheat Co., located at Niagara Falls, N. Y. The following data on this test were taken from a paper by Mr. W. A. Slater, presented before the recent meeting of the American Concrete Institute:

The building is of reinforced concrete construction of the flat-slab type. It is three stories in height, and is divided by the columns into panels 20x22 ft. The test was made on the first floor above the basement story. This floor is 7 ins. thick in the central portion of the panel, 9 ins. thick throughout an area 8 ft. 6 ins. square surrounding each column, and is designed to carry a live load of 125 lbs. per square foot. The columns are octagonal in shape, those of the basement story being 25 ins. in the short diameter and ending at the top in 3-ft. 6-in. flared heads. The columns of the first story are 23 ins. in the short diameter. The reinforcement, all of which is placed in the two directions parallel to the sides of the column, is designed to resist positive moment at the centers of all spans and negative moment at the columns and across the edges of all panels.

Originally nine panels were loaded with 191 lbs. per square foot, after which the load was removed from six panels, leaving three adjacent central panels loaded. These three panels lie in a row and are one span removed from the exterior wall of the building. Finally, the load was removed from all except one panel, the central panel of the entire test area, and the load on this panel was increased to 243 lbs. per square foot.

Deformations were measured at 288 gage lines, and the results were very consistent among themselves. It was found that the condition of the three adjacent panels loaded gave steel stresses at the center of the span more than 50 per cent greater than with nine panels loaded, and that the stress at the support was about 20 per cent less with three panels loaded than with nine panels loaded. It seems apparent that the tendency of the floor to deform as a cylinder instead of as a saucer, its action as a continuous beam and the lateral distribution of stress to the unloaded portions of the slab, all contributed to these differences between the stresses un-



Note: Diagram Based on Formula, 16000 - 70 $\frac{L}{r}$, and Strength about Axis Perpendicular to Web. E. & C. When Column Carries Combined Axial and Eccentric Load use Value of (e) in Diagram = Actual Eccentricity × $\frac{W}{P+W}$

Fig. 1. Diagram for Determining the Required Area for Plate-and-Angle Columns Eccentrically Loaded — Diagram Also Applicable to Axially Loaded Columns.

der a nine-panel load and those under a three-panel load. The results of the test seem to check the conclusion arrived at analytically, namely, that the greater portion of the unbalanced moment was carried by the columns and very little by the slab. This conclusion is somewhat in conflict with the observation that there was so great an increase in the stress at the center of the panel when only three panels were loaded. In this respect it is typical of the generally observed fact that the results obtained from such tests are usually indicative of the general trend of results, but cannot be applied too rigidly as final conclusions.

Results of Experiments to Determine the Effect of Electrolysis in Reinforced Concrete, Possibilities of Trouble in Actual Structures, and Some Effective Protective Measures.

The subject of electrolysis in buildings is one which has caused considerable discussion and about which there is much uncertainty. Although there has been apprehension concerning the possible danger to reinforced concrete structures due to stray currents from electric railways and from other power sources, there are few authoritative data as to the actual effect of electric currents on such structures. Realizing that more information on this subject was needed the United States Bureau of Standards has made an elaborate series of experiments to determine the effect of electrolysis on reinforced concrete. The results of these experiments and the conclusions drawn from them have been published as Technologic Paper No. 18, "Electrolysis in Concrete," by E. B. Rosa, Burton McCullum and O. S. Peters. This paper contains 136 pages of tabular data and discussion. An abstract of it, omitting the tabular data, but including statements as to the character and scope of the tests and the conclusions drawn from them which are of direct interest to engineers and builders, is herewith given:

At the time that the experiments were begun in 1910, the following facts had seemingly been established by the work of other investigators:

(1) That, as far as the experiments went, the passage of an electric current of whatever magnitude from embedded iron to concrete in the presence of moisture resulted in cracking of the concrete and rusting of the embedded iron. These cracks extended radially from the iron and opened out gradually as the test progressed until the block in which the iron was placed would go to pieces or could be easily broken.

(2) The passage of an electric current from concrete to embedded iron resulted in no damage whatever to the block, except in one or two extreme cases, where the voltage used was very high. This led to the conclusion that no damage is to be expected where current is found flowing from concrete to iron—that is, where the iron is cathode.

(3) The specimens of concrete showed a tendency to protect the iron contained in them by virtue of a large increase of resistance as the test proceeded. This increase is not entirely permanent, as is evidenced by measurements taken before shutting off the current, and again some time after stopping the current, and is found in greatest degree where embedded iron is anode—that is where the current passes from iron to concrete. Where the iron is cathode, the rise of resistance occurs, but is not nearly so great. There is, however, no instance in any previous investigation of this increase of resistance being sufficient to reduce the current to a negligible value.

(4) The voltage apparently had nothing to do with the phenomena observed beyond affecting the magnitude of the current and consequently the rapidity of the action.

Beyond these four points there appeared to be little agreement among the various investigators. Especially contradictory were the theories advanced to explain the phenomena observed. As the result of experimental evidence of record, the following different theories had been advanced to account for the destruction of the concrete:

(1) Gas Pressure: When an electric current passes between electrodes in an electrolyte, more or less gas is evolved at the electrodes, and it was held that this gas, liberated within the mass of the concrete, escaped with such difficulty that sufficient pressure was developed to crack the concrete.

(2) Heating: The passage of an electric current through concrete generates heat therein, and if the current density is high the concrete may become quite hot. It was shown that local heating by a flame or other means would produce cracking, and the conclusion was therefore drawn that the cracking of the concrete under the influence of the electric current was due to its heating effect.

(3) Electrochemical Deterioration of the Concrete: The early observations of Toch, Langsdorf and others led them to believe that the concrete underwent a marked chemical or physical change, whereby it became soft and fragile, and that this apparent decay of the concrete was in large measure responsible for the cracking observed.

(4) Mechanical Pressure: When a current passes from iron into the concrete, corrosion of the iron results with the ultimate production of insoluble compounds of iron, largely the oxides. These form near the surface of the iron and occupy about 2.2 times as much space as the original iron from which they were formed. It is evident, therefore, that considerable mechanical pressure might be developed in this way between the surface of the iron and the concrete, possibly sufficient to produce the cracking observed. This theory had perhaps been most widly accepted, but it was directly opposed by Upson and Barker as a result of their experiments with collapsible electrodes. When these electrodes were used, the concrete was found to crack in very much the same manner as when solid electrodes were employed, and it was therefore concluded that there was no appreciable pressure developed on the surface of the electrode. Upson and Barker advanced the theory that the cracking was due to the formation of deposits of iron compounds, chiefly hydrates, within the pores of the concrete, and these on oxidation developed the forces which are responsible for the cracking of the concrete.

In addition to the laboratory investigations a number of cases had been reported in the technical press in which damage of a serious nature had occurred to certain concrete buildings, bridges, and other structures in which the damage was attributed to stray currents from power circuits.

The investigation discussed in this paper consisted of three parts, as follows:

(1) Laboratory investigations relating to the nature and cause of the phenomena produced by the passage of electric currents through concrete.

(2) Investigations in the field with the view of establishing the probable extent of the danger in practice and the circumstances under which trouble is most likely to occur.

(3) A study of the various possible means of mitigating trouble from this source, leading to specific recommendations.

Laboratory Investigations.

Preliminary Experiments.—At the outset of the investigation a number of the experiments described by previous experimenters were repeated in order to verify their results and also to afford an opportunity for studying at first hand the phenomena previously observed. At the same time numerous modifications of these experiments, as well as many radically different ones, designed to throw light on particular phases of the subject, were instituted. In this preliminary work one general type of specimen was adhered to as far as practicable. A number of well-known brands of Portland cement, all of which conform to the requirements of the standard specifications for Portland cement, were selected together with sand and stone similar to those used in large quantities by builders in the city of Washington for reinforced concrete construction work. Using filtered Potomac River water, these ingredients were made into a 1:2½:4 concrete, proportions which gave a fairly dense aggregate. In many experiments, especially where chemical analyses were to be made, quartz sand and distilled water were used, as noted later. The specimens were molded into cylinders 6 ins. in diameter and 8 ins. long, with an electrode embedded in the axis of each cylinder.

The specimens were allowed to remain in the molds until set sufficiently for handling and then were removed and buried in wet sand for approximately 20 days. At the end of this time they were taken out of the sand and placed in an inclosed case, where they were wet down occasionally until used.

Where iron served as material for the embedded electrode it was cleaned of scale and rust by pickling in dilute sulphuric acid. After the pickling process was completed the iron was removed from the acid, dipped in limewater and immediately embedded in the specimen, after being numbered and weighed. This insured the entire absence of rust to begin with. Direct electric current for the experiments was obtained from two sources. One of these was the 115-volt power circuit of the Bureau and the other the 15-volt circuit of a small motor-generator set. The wires from these circuits were led into cases containing the specimens under test and terminated in binding posts at convenient points. The currents used by the specimens were very small. The description of the phenomena observed is given under two heads, viz., anode effects, or those effects observed where the current flows from the embedded iron into the concrete and cathode effects, or those occurring when the current flows from the concrete to the embedded iron. In each case experiments were carried out both under high and low voltage, and as the results were radically different in the two cases, they are discussed below under separate heads. In the high-voltage tests the specimens were subjected to voltages varying from about 50 to 70 volts, and the current flow was continuous, except for infrequent and irregular intervals when work was being done on the specimens under test or on the electric circuits to which they were connected.

ANODE TESTS ON HIGH VOLTAGE.

Only a comparatively small number of specimens were tested on high voltage, because such conditions are abnormal and would rarely, if ever, be encountered in practice. A few of these were made, however, to check the observations of previous experimenters, and the results noted in these tests are recorded briefly below. In the preliminary tests on high voltage, in which iron served as the anode, eleven specimens were used. They were all of like proportions, viz. 1:2½:4, and were connected two in series on 115 volts direct current. All of the specimens behaved in substantially the same manner. During the first few hours there was an appreciable rise in the external temperature. In most cases this rise amounted to from 12° to 25° C. above room temperature. This rise of temperature usually reached its maximum and diminished very considerably before any damage to the concrete was apparent. After a few hours of current flow bubbles of water containing iron rust began to appear around the anode and cracks soon developed in the concrete. The manner in which the cracking occurred was the same in all specimens. First there appeared a very fine crack, beginning at the iron and extending outwardly in practically a radial direction. This crack soon extended to the outer surface of the concrete cylinder, and then gradually widened while at the same time two or three other smaller cracks would appear on the opposite side of the anode from the initial crack, also radiating from the center.

The manner in which these cracks occur and develop is strongly suggestive of a wedge-like action slowly applied at the center of the specimen (see Fig. 1). After the first crack has extended from the center to the outer surface of the concrete, the specimen can readily be pried open with a screwdriver and separated into several pieces. The individual pieces of concrete thus obtained are, however, to all appearances as strong as similar specimens of concrete that have not been subjected to the influence of electric currents and are broken with a hammer with the same difficulty. There is no indication whatever that the cement decayed or deteriorated in any way, as reported by some earlier investigators, but, on the contrary, the appearance of the concrete and its mechanical properties both indicate very strongly that there has been no such action.

On breaking open the specimens the embedded iron was found to have been very badly corroded, thick layers of scale consisting largely of the oxides of iron having formed all over the surface and to some extent in the voids in the concrete adjacent to the iron. In those specimens which were left in circuit for a long period after cracks developed the deposit of oxides spread for some distance out into the cracks. Upon breaking open the solid portions of the concrete it was found that the discoloration due to the products of corrosion had not penetrated the body of the concrete more than about ⅛ in.

ANODE TESTS ON LOW VOLTAGE.

Tests on specimens subjected to 15 volts and less were carried out in much greater number than those on high voltage. At the utset of the investigation 90 specimens containing iron electrodes were placed in circuit and watched for a period of about 5,500 hours (7½ months), during which time careful records were kept of voltage and current flow. Now and then a few specimens were broken open to give an idea of what was taking place. At the end of the time mentioned a large number of specimens were broken open and examined, the amount of the corrosion determined, and the general condition of the concrete noted. A most conspicuous feature of the results of this test, and a very surprising ne in view of the results previously obtained t higher voltages, is the fact that cracking lmost universally failed to occur. Of the 90 pecimens under test only three had cracked t the end of 5,500 hours.

It seems allowable, therefore, to say that ot one of 87 normal specimens made of ordinary Portland cement and put under test on 5 volts or less had cracked at the end of ,500 hours. Of the specimens which were hen opened for examination it was found hat in every case the concrete was broken with difficulty and appeared to be as sound s that of similar specimens made at the ame time and not subjected to the action of lectric currents. In practically all cases there vas more or less corrosion around the anode t the point where it entered the concrete and n some instances for an inch or so below the urface. In some cases, also, where there vere voids in the concrete next to the iron here was some rust and slight pitting, but in early all cases where the concrete was in ctual contact with the iron the corrosion was uite small, and in many cases the iron was ractically as bright and clean as when placed n the concrete.

It is important to note that the total number of ampere-hours per square inch of embedded electrode surface carried, on the average, is considerably larger than the corresponding figures for the high-voltage specimens at the time they cracked, the average eing 2.6 for the low-voltage specimens, as gainst 0.83 required to crack the high voltage pecimens. It is evident, therefore, that the uantity of electricity that passes through a pecimen does not alone determine the mount of damage that it may do, but that he rate at which the current flows is also n important factor. Moreover it must be vident from these observations that the rate t which damage occurs decreases with decreasing voltage much more rapidly than the oltage is lowered, since in the present instance a reduction of the voltage to one-fourth of the value used in the high voltage tests enabled the specimens to run with little r no damage for a period over 200 times as long as was required to crack and split open the specimens on the higher voltage.

The average resistance of the specimens at the beginning of the test was 91 ohms, and at the end of the test it had risen to 12,500 ohms, or about 137 times its initial value. It is obvious that the rise of resistance, and therefore the tendency toward self-protection is much more marked in the case of the low-voltage specimens than in those run on the higher voltage.

Experiments were also made using copper, aluminum, brass, carbon and copper-clad steel as anodes, but the results will not be discussed here.

CATHODE EFFECTS USING IRON AND OTHER METAL ELECTRODES.

In the preceding sections the phenomena noted are those resulting when current flows from the embedded iron or other metal out into the concrete. When the direction of current flow is reversed, making the iron cathode, very different effects are produced. In this case there is no tendency for the iron to corrode because of the current flow, but on the contrary the iron is protected from any natural corrosion that might tend to take place. In the published work of previous investigators no mention is made of any injurious effects, either to the iron or the concrete in those specimens in which the current flowed from concrete to iron, except in a single instance in which the specimen cracked. The cracking in this isolated instance was beyond doubt merely incidental, since all other investigators have failed to note any tendency to crack when the iron is made cathode. The conclusion has therefore been widely accepted that when the current flows from concrete to iron no effects are produced, and at the time these experiments were begun there appeared to be no substantial ground on which to question this conclusion. It was deemed advisable, however, to confirm these observations, and accordingly a number of specimens were made exactly similar to those used in the anode tests above described, using not only iron but also brass, copper, carbon, and aluminum as electrodes. These were placed in circuit with the current flowing from the concrete to the electrode. Both high and low voltages were used, and the conditions in general were kept exactly the same as with the anode tests except for the direction of current flow. It was noted that there was a rise in resistance as the test proceeds very similar to that which occurs in the anode specimens, but the rise is less marked, the average ratio of increase being about 10 to 1. The first four specimens contained iron electrodes. Throughout the test water was forced out around the embedded iron and kept the top of the specimen wet. This forcing out of water was evidently due to the formation of gas at the cathode, which on escaping forced the water through the pores of the concrete to the surface. The water carried calcium hydroxide in solution, and as evaporation took place calcium carbonate was deposited in rings surrounding the cathode on the surface of the concrete. The greater part of the gas evolved was hydrogen. As the test proceeded the mortar on the top surface of the specimen immediately surrounding the cathode became quite soft. This softening extended in the surface to a distance of about ¾ in. from the cathode, and in this region the concrete was darker, particularly while wet, than elsewhere. After the expiration of several months, and at intervals thereafter, certain of the specimens were broken open for examination. In every case the concrete blocks were broken with difficulty, the main body of the concrete being apparently as sound as in similar specimens not subjected to the action of electric currents. On laying the specimen open it was found that the embedded metal was in a perfect state of preservation, but the entire region surrounding the cathode for a distance of 1/16 in. to ¼ in. from the surface of the metal was considerably darker in appearance than the main body of the concrete and was very soft, like the concrete immediately surrounding the cathode at the surface (see Fig. 2). The cement here could be shaved off with a knife like soft soapstone.

In all of the above specimens there was a rather sharp line of demarkation between the softened area and the remainder of the concrete, the soft portion being in every case readily distinguishable by its darker color. Outside this darkened zone the concrete appeared to be as sound as in specimens not carrying current, and no physical change of any kind could be detected therein. After the specimens had been broken open for some time and allowed to dry, the darkened zone became somewhat lighter in shade, but there always remained a distinct difference readily detectable by the eye. The softening also diminished greatly as the specimen became drier, and after becoming thoroughly dry it became nearly as hard as the unaffected mortar, but remained distinctly more friable.

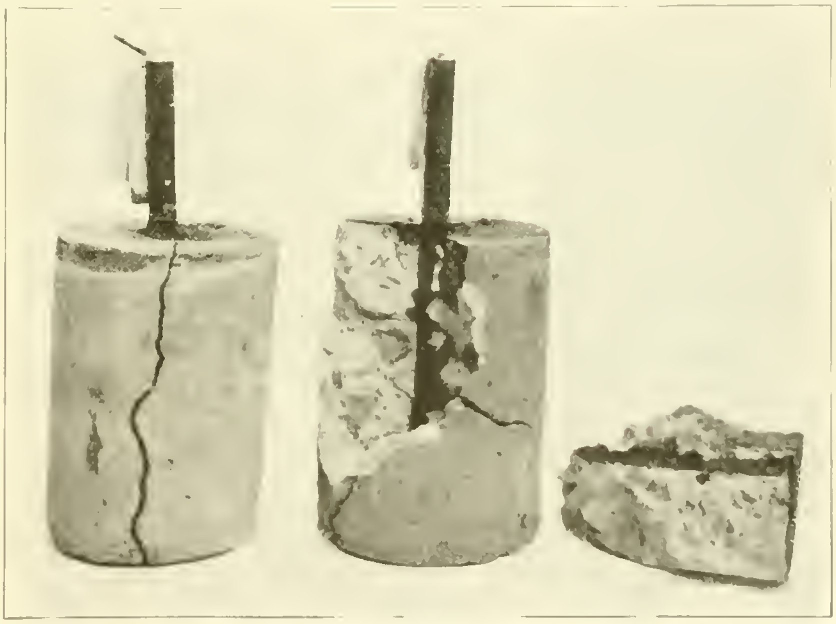


Fig. 1. Effect of High Voltage Current Flowing From Embedded Iron to Concrete (Anode Test).

BOND TESTS OF CATHODE SPECIMENS.

The disintegration of the mortar in the immediate vicinity of the cathode, as described above, led to tests on the relative strength of the bond where current had passed with embedded iron cathode and where it had not. For this specimens with ¾-in. iron rods were used, together with four identical specimens through which no current had passed. By means of an Olsen testing machine the rods were pushed through the specimens a sufficient amount, about 0.1 in., to give the maximum bond strength and also the friction load after the bond was broken.

The passage of current under the conditions of the test reduced the bond to about one-fifth of its original value.

These tests indicated that at least during the first few months of a test the amount of the disintegration is probably roughly proportional to the quantity of electricity that has passed through the concrete, and is thus but indirectly affected by the voltage used, at least within the range of voltages used in these tests. In this respect it is in marked contrast to the anode effects previously described, which appear to diminish much more rapidly than the voltage, until at voltages of 5 to 10 in the specimens used they practically disappear.

EFFECT OF ELECTRIC CURRENTS ON THE MECHANICAL STRENGTH OF NON-REINFORCED CONCRETE.

While the foregoing experiments appear to show quite clearly that no effect is produced by the passage of an electric current through the main body of the concrete remote from the electrodes, it was deemed advisable to corroborate this with a series of mechanical tests on specimens so designed as to eliminate the electrode effects.

In order to secure further information on this subject a test was carried out using concrete cylinders 6 ins. in diameter and 8 ins. long with no iron embedded. These cylinders were made of 1:2½:4 concrete, using Old Dominion cement for part of them and Alpha for the rest. Sand and crushed trap rock constituted the aggregate. After the concrete had become thoroughly set 18 of the cylinders were provided with electrical connections by plastering a 3x3-in. carbon plate to the ends of each cylinder with cement mortar. A copper lead was attached to each carbon plate. The potential gradient impressed on these specimens was 172 volts per foot and the test was extremely severe. Current flowed continuously for 11 months. Approximately 75 ampere hours of electricity passed through each specimen under a potential gradient of about 14 volts per inch. The current density averages about 0.0003 ampere per square inch of cross sectional area of the specimen. At the end of a period of 11 months the specimens were tested to determine their crushing strength. Nearly all of the failures showed the usual 45° shear. There is no indication that the passage of current affected the crushing strength of the concrete.

CAUSE OF CRACKING OF REINFORCED CONCRETE BY ELECTRIC CURRENTS.

As regards the various theories that have been advanced as to the cause of cracking of reinforced concrete which were previously mentioned in this paper, the first, that of gas pressure, appears to have little evidence to support it. In all cases it has been found that a much greater volume of gas is liberated at the cathode than at the anode, and if the pressure due to any accumulation of gas within the concrete were an important factor in causing the cracking, we should expect to find even greater damage in cathode specimens than in anode specimens. On the contrary, however, the cracking is found to be peculiarly an anode effect, and it therefore seems certain that gas pressure at most plays but a very minor part in causing the damage. As for the second theory, that of heating, the same objection applies, because the heat developed in the cathode specimens is substantially the same as in the anode specimens under similar conditions. It must be admitted that heating of the embedded iron, if sufficiently great, is capable of developing cracks in concrete, but with the ordinary circumstances under which cracking has occurred in the laboratory this is not sufficient to contribute materially to the damage, as the cathode experiments show.

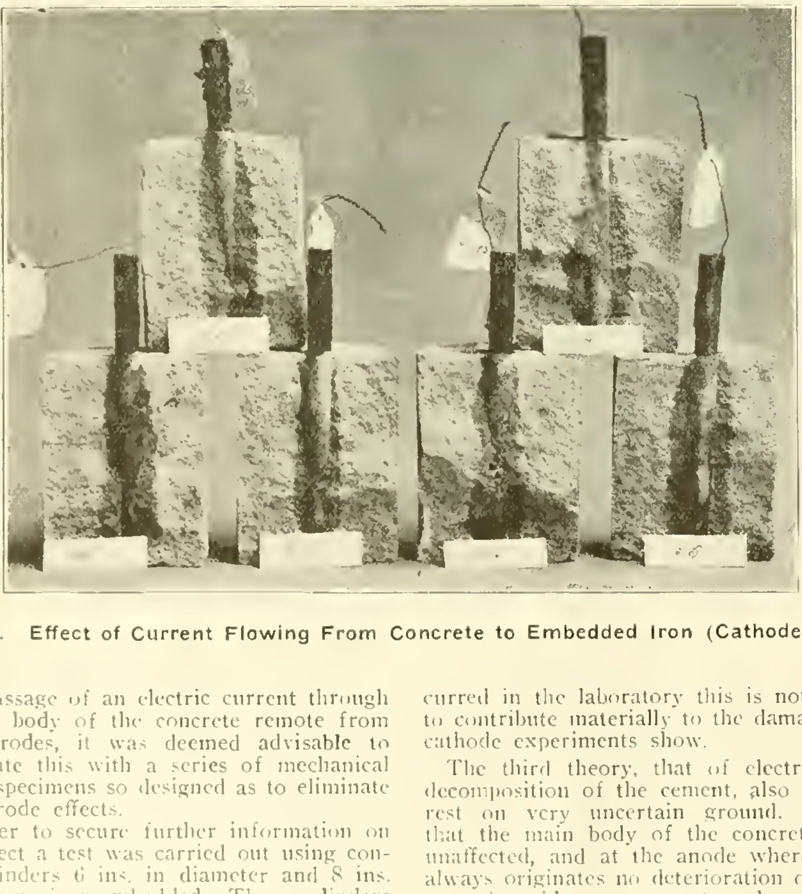


Fig. 2. Effect of Current Flowing From Concrete to Embedded Iron (Cathode Test).

The third theory, that of electro-chemical decomposition of the cement, also appears to rest on very uncertain ground. The fact that the main body of the concrete remains unaffected, and at the anode where cracking always originates no deterioration of the concrete is evident, argues strongly against this theory. It appears inevitable therefore that we must conclude that the cracking is a direct result of some process occurring at the surface of the anode, and hence we must look to the anode phenomena for the true explanation of the damage. The explanation previously advanced that the cracking is due to the deposition of oxides of iron between the surface of the iron and the concrete, which, occupying a larger volume than the original iron, produces a mechanical pressure sufficient to crack the surrounding concrete, appears on the surface to be well founded. There are, however, no published data showing that such pressures actually develop or indicating what magnitudes they may attain.

MEASUREMENT OF FORCE PRODUCED BY CORROSION OF IRON IN CONCRETE.

Another and more striking experiment to demonstrate the existence of a mechanical pressure and at the same time to give an idea of the order of magnitude of the forces developed was made with the apparatus shown in Fig. 3. Several sections of 4-in. cast-iron pipe about 5 ins. long were cut through on one side with a hack saw, making slits about 1/16 in. in width. These sections of pipe were then filled with concrete and an iron rod embedded in the center of each to serve as one electrode. After being properly aged, the specimens were connected in circuit, with the outer cast-iron sleeves as anodes. In that case corrosion would take place on the inner surface of the cast-iron and produce a layer of oxide between the iron and the concrete. If any considerable pressure were produced it would result in a tendency to expand the cast-iron sleeve, with consequent widening of the slit. This actually proved to be the case. The increase in the width of the slit in this case is about 3/32-in. As the cast-iron sleeve was about ½ in. thick and possessed considerable rigidity, a very appreciable mechanical pressure is here made evident. Calculation showed that the natural expansion of the concrete could not have produced more than a few per cent of the effect observed. One experiment showed a radial pressure of about 2,400 lbs. per square inch at the surface of the electrode, mostly due to the formation of rust; another experiment showed 3,760 lbs. per square inch; and still another 4,700 lbs. per square inch. It is thus seen that the force produced is sufficient to destroy the concrete.

RISE OF ELECTRICAL RESISTANCE OF REINFORCED CONCRETE DUE TO FLOW OF CURRENT.

The electrical resistance of concrete is a factor of great importance, but one which varies so greatly with varying physical conditions, especially with varying moisture content that it is difficult to give any reliable numerical values except in the case of approximately saturated specimens. An average figure for representative specimens of thoroughly wet concrete may be taken as varying from 4,000 to 6,000 ohms per centimeter cube, depending somewhat on the proportions. With reduced moisture content the resistance rises rapidly, the concrete becoming a fairly good insulator when thoroughly dry.

A notable exception to the practically general rule with regard to the increases of resistance of reinforced concrete when current flows through it will now be considered. It is not an uncommon practice in placing concrete in cold weather to dissolve 2 to 3 per cent of common salt in the water used in making the concrete. The effect of this addition of salt to concrete upon its electrical properties appeared very important, and tests were therefore made to determine this effect. In all cases where such specimens were tested as anodes the damage was not only found to be very much hastened, but instead of the usual increase of resistance with the passage of the current the resistance actually decreased in some cases as the test proceeded. Two specimens which were made up with no addition of salt, but which were tested in a 3 per cent salt solution decreased from 115 to 80 ohms and from 130 to 60 ohms, at the end of 410 and 1,490 hours, respectively.

A second test, in which the concrete was made up with a 10 per cent salt solution and tested as anode on 15 volts over a period of five days, showed a decrease of resistance of 25 per cent. Cathode specimens tested in a manner similar to the tests on the first two specimens mentioned showed an increase of resistance of 20 to 25 per cent in the same length of time. This marked difference in character from that of all the other specimens tested in this work must be attributed chiefly to the salt, or rather to the presence of an excessive amount of the acid radical chlorine. The currents taken by the specimens were rather high in most cases, and hence heating may have had something to do with the decrease of resistance instead of an increase but it can not account for all of it. In tests on specimens to which no salt was added the temperature rise was nearly as great, but the rise of resistance never failed to occur.

RISE OF RESISTANCE OF CONCRETE BURIED IN DAMP EARTH.

It is evident that a rise of electrical resistance of reinforced concrete takes place under all the conditions of the electrolysis experiments conducted in the laboratory except when an appreciable amount of chlorine, or perhaps other acid radical, is present in the cement. These laboratory conditions provide a liquid electrolyte in all cases. In practice wet soil would more often be the electrolyte if conditions were favorable to electrolysis. Current would also be likely to flow between sections of reinforcement without leaving the concrete. In order to determine whether or not a rise of resistance would occur in the case of concrete in soil, two blocks of concrete containing embedded iron were molded in holes dug in the earth so that about 3 ins. of the block projected above the surface. The blocks were of 1:2½:4 concrete, 3.5 ft. square x 2.5 ft. deep. Current flowed continuously except for about 33 hours each week. A rise of resistance was found to take place, but it appeared in a longer time than in the case of specimens tested in jars.

RISE OF RESISTANCE OF CONCRETE SETTING IN AIR.

A similar test to that above described was made by passing the current through three reinforced concrete beams 5 ins. square x 5 ft. long. These beams were built of 1:2½:4 concrete, and were reinforced with ¼-in. rods. The rise of resistance was found to occur in much the same manner as in blocks buried in the earth.

In order to gain some information as to the specific resistance of concrete while setting in air without current flowing, eight 1:2½:4 concrete specimens were made in the form of rectangular blocks. When the blocks were removed from the molds the electrical resistance between the plates placed in them were measured by alternating current. The specimens were then buried in damp sand for about eight weeks. On removal from the sand the resistance was measured again, after which the specimens were placed in air and the measurements repeated from time to time. The weight of the specimens remained nearly constant from the time of removal from the sand to the end of the test. This can best be accounted for, in view of the large increase of resistance noted, by the fact that with the slow-setting process which goes on during the first few months of the life of concrete a considerable amount of water is taken up by hydration, enough, possibly, to counteract the drying effect of the atmosphere.

It is therefore seen that a great rise of resistance takes place under conditions practically identical with those occurring in practice, and it is, therefore, an important factor in reducing damage to concrete structures, and particularly so in the case of moderate or low voltages, as a few volts or less, such as would almost invariably be met with under actual conditions. Its magnitude is such as to increase by many times the life of a structure over what it would have if subjected to severe electrolytic conditions and the rise of resistance did not occur.

VARIATIONS IN MAGNITUDE OF ELECTROLYTIC CORROSION OF IRON IN CONCRETE

It is well known that in cases of corrosion by electric currents the total amount of corrosion does not in general correspond with Faraday's law, but as in many cases considerably less than the theoretical amount. In particular, in the case of concrete, it was to be expected that the actual amount of corrosion would be less than that expected from Faraday's law, because of the alkalinity of the electrolyte. This obviously has an important bearing on the subject of electrolysis in reinforced concrete, because the extent of the reduction in the amount of corrosion determines the damage which will result to the reinforcing material by a given ampere-hour discharge, and also determines in greater or less degree the extent of the injury to the concrete itself.

Effect of Temperature on Efficiency of Corrosion.—One of the most important facts to be noted in connection with the efficiency of corrosion in concrete is the great change in the efficiency of corrosion attendant upon a change of the voltage through a wide range of values and the consequent accompanying change of current density at the surface of the electrode. There is evidence of this change in the efficiency of corrosion in the results of the anode tests.

Tests were made on several different brands of cement, three cubes being made of each brand. One cube was kept without current as a check on natural corrosion, and the other two were tested electrically. The check specimens showed very little natural corrosion, the losses in these cases being hardly greater than the errors which would be expected to occur in weighing. The efficiencies of corrosion show beyond a doubt that under the conditions of these experiments but a small fraction of the current passing through a specimen of reinforced concrete is effective in dissolving the iron. The variations in efficiency of corrosion between different cements are quite marked, the corrosion efficiency of some specimens being three or four times that of others.

For specimens of the same form the various current densities were in practically the same ratio as the voltages at like periods of the tests. The decrease in the efficiency of corrosion therefore does not take place in the same proportion as the decrease in current density. For the type of specimen used in these tests and for the conditions of the tests it seems that corrosion practically ceases at a voltage somewhat below 15 volts. The comparison of these results, together with the fact that where the high efficiencies of corrosion prevailed there was always high temperature, led to other tests which were designed to show the effects of temperature on the efficiency of corrosion.

For the purpose of carrying out the temperature tests a number of cylindrical specimens 4 ins. in diameter by 4½ ins. long were made up of a 1:2 motar consisting of quartz sand and Old Dominion cement mixed with distilled water, the sand having been washed with tap water before using. A ½-in. round iron electrode was embedded to a depth of 4 ins. in the axis of each cylinder. After being removed from the molds the specimens were allowed to set in a damp closet two weeks before testing.

The first series of tests was carried out, using varying current density and constant temperature. The object of this test was to ascertain whether or not any variation in efficiency of corrosion might be directly due to change in the current density over the surface of the electrode. The only temperature which could be relied on for constancy in the specimen was boiling temperature, which for the electrolyte, was a trifle more than 100° C. Seven different current densities were used, the current density for each set of specimens being kept constant throughout the test.

The results show that the current density in the ratio of 1 to 40 causes no material change in the efficiency of corrosion. For a given current density there are variations in the efficiencies of corrosion between individual specimens, but these variations may be attributed to variations in the concentrations of the dissolved substances of the electrolyte.

A second series of tests, the purpose of which was to show the effect of variation of temperature on the efficiency of corrosion, was conducted in a manner similar to that of the preceding series of tests. The range of temperature was from 3° to 100° C., the temperature of the electrolyte being the one measured and recorded. In order that the heating effect of the current on the interior of the specimen might be nearly the same at all temperatures the current was kept constant and equal to 0.030 ampere through the four specimens of each set in series. The total average voltage required for this was about 26 volts. Since heat was being constantly lost from the specimen through evaporation of water such a rate of heat liberation distributed as it was between positive and negative electrodes could not result in a material increase of the temperature of the interior of the specimen over that of the electrode. Below 50° the efficiency of corrosion is very small. Since current density has been shown to have no direct effect on efficiency of corrosion, the great difference in efficiency of corrosion between high and low voltage specimens must therefore be attributed to the heating effect of the current. If the heating effect of the current is sufficient to raise the internal temperature of the specimens to 50° C. or more, active corrosion occurs. Below 50° the iron remains passive, or nearly so. This has been found to be strictly true, however, only for normal concrete specimens to which no foreign ingredient of marked corrosive tendencies has been added, as will presently appear.

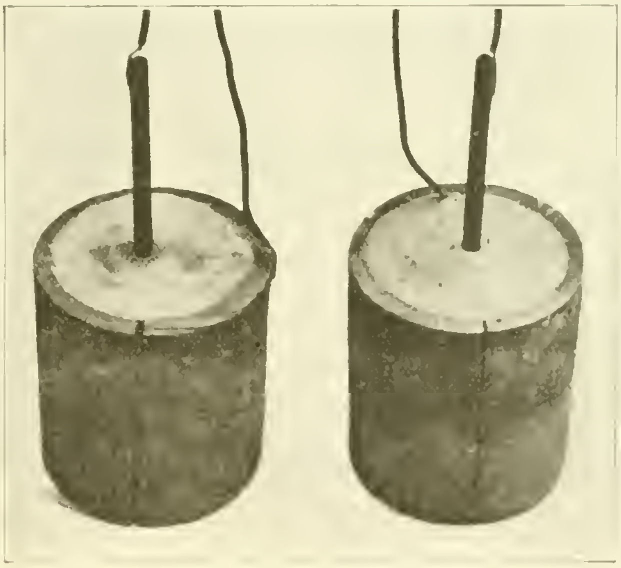


Fig. 3. Apparatus for Determining Force Produced by Corrosion of Iron in Concrete.

Effect of Waterproofing Compounds.—Materials are often added to the concrete to make it more impermeable to water. Hydrated lime and a number of proprietary compounds, consisting of powders, pastes and liquids, are sometimes mixed with the concrete. A number of specimens were made up and tested, using a number of these ingredients with a 1:2½:4 concrete. These were added to the cement and did not replace it. The addition of some of the ingredients did not result in a very material increase in corrosion, while others caused a very rapid corrosion.

Effect of Salt and Calcium Chloride.—The effect of sodium chloride and calcium chloride proved to be remarkable. The addition of 0.02 per cent of $Ca\ Cl_2$ to the cement resulted in a very marked increase in the efficiency of corrosion. Below that percentage the addition of it did not seem to have an appreciable effect. An addition of ⅓ per cent of sodium chloride was found sufficient to destroy the passivity of iron almost completely. A comparison of Figs. 2 and 4 shows the effect of an addition of sodium to the concrete. It thus appears that great care should be exercised in selecting an ingredient to add to reinforced concrete for any purpose whatsoever, provided there is any likelihood that the structure will later be subjected to the action of electric currents. In such cases the addition of chlorine should be avoided entirely, and before using it an integral waterproofing should be carefully examined for the presence of any soluble salt having a marked tendency to increase corrosion of the iron.

EXPERIMENTS WITH PROPOSED METHODS FOR MINIMIZING ELECTROLYSIS IN REINFORCED CONCRETE.

A number of methods have been proposed by various investigators for preventing or reducing damage in reinforced concrete by electrolysis. Little has been known as to the efficacy of these proposed remedies, however, and in order to throw further light on the questions regarding their practical value a number of laboratory experiments were instituted. These methods are, in general, only applicable to structures which are in process of erection.

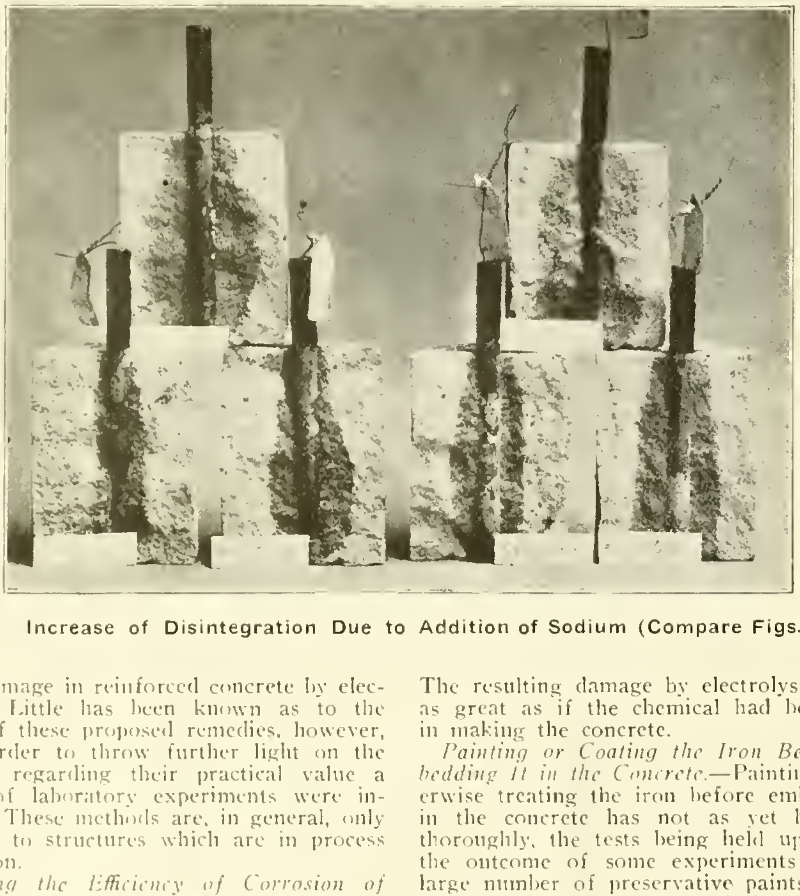


Fig. 4. Increase of Disintegration Due to Addition of Sodium (Compare Figs. 2 and 4).

Reducing the Efficiency of Corrosion of Iron by Chemical Means.—Up to the date of this writing the attempts toward reducing the efficiency of corrosion by chemical means have met with very indifferent success. In an endeavor to contrive some means for reducing the efficiency of corrosion of iron when made anode in concrete a number of substances known to retard the natural corrosion of iron in their aqueous solutions were tried out. The resulting efficiencies of corrosion are extremely small in all cases, while the differences between those obtained with the chemicals added and those without any added are not of a magnitude which would suggest their use in practice as preventives of electrolytic damage.

It is well known that the presence of sulphates in even comparatively small amounts tends to destroy the passive state which iron generally assumes in strongly alkaline solutions, and, accordingly, a trial was made with barium hydrate added to the cement in the form of a fine powder. An examination of the results, however, did not reveal any evidence from which we can conclude that the addition of barium hydroxide is of any material value in reducing the efficiency of corrosion. In the case of the iron-ore cement there appears to have been a decrease in the corrosion, but in the case of other cements the reverse is the case.

It should perhaps be noted in this connection that the barium hydrate added to the cement would have no effect on the activity of any soluble chlorides that might be present.

Under ordinary conditions and at usual temperatures the efficiency of corrosion of iron in normal concrete is very low; it seems that efforts might most profitably be directed toward keeping it low by preventing the addition to the concrete of any acid radical of marked corrosive tendencies in a form which is readily dissolved and ionized. The wisdom of such a precaution is indicated in the section dealing with electrolytic corrosion of iron in concrete and should be taken, not only with reinforced concrete in the process of making, but at every stage of its existence. Concrete takes up a very appreciable amount of water, and when a salt or an acid in solution comes in contact with its surface the chemical may diffuse throughout the mass. The resulting damage by electrolysis may be as great as if the chemical had been added in making the concrete.

Painting or Coating the Iron Before Embedding It in the Concrete.—Painting or otherwise treating the iron before embedding it in the concrete has not as yet been tried thoroughly, the tests being held up to await the outcome of some experiments with a large number of preservative paints for iron as preventives of natural and electrolytic corrosion in the presence of air and moisture. Three standard size specimens were made up for one test along this line, however, using ¾-in. round iron electrodes which had been dipped in melted pitch before embedding them. They were connected up as anode on 15 volts and left in circuit more than one year. The currents in two of the specimens were inappreciable at all times. The third carried a current which varied a great deal, but the highest current reading at any time was only 20 mil-amperes. It is evident, therefore, that the pitch had considerable effect in preventing current flow, but in common with all paints used in this manner it has the disadvantage that no bond forms between the concrete and iron when the concrete sets. This limits the application of such a method of protection to structures where the strength of the bond is not an important matter.

Tests of Integral Waterproofing.—In a search for a solution for the problem of increasing the specific resistance of concrete a trial was made of a number of integral waterproofings for concrete which are commonly sold in the market. These integral compounds are sold in the form of powders, pastes and liquids and are designed for incorporation within the mass of the concrete while mixing the aggregate and their intended function is to reduce the porosity of the mass or otherwise render it impermeable to water. The method of application varies with the character of the compound; the procedure generally being to mix the powders with dry cement, the pastes and liquids with the water used in making the concrete.

The concrete specimens used for the tests were made of a 1:2½:4 mixture, with ¾-in. round iron electrodes embedded in them. Each compound was added to the concrete in the manner prescribed by its manufacturer. After the specimens had been taken out of the molds they were allowed to set in water or wet sand for eight or ten weeks before testing.

In making an interpretation of the results of the tests on integral waterproofings the tendency of normal reinforced concrete to protect itself against the effect of electric currents should be considered. In the section dealing with the rise of resistance of reinforced concrete due to flow of current it is seen that when saturated specimens are subjected to the action of current the rise of resistance is sufficient within a few weeks to prevent damage unless the conditions of electrolysis are extremely severe. Therefore, unless a waterproofing integral causes a decrease in the conductivity of concrete under the conditions of application in practice to an amount comparable with that observed in connection with the flow of current through normal reinforced concrete its use would not be justified merely on the ground of preventing electrolysis. That is, in order to be efficient in the prevention of electrolysis the addition of the waterproofing agents should cause a reduction in the conductivity of wet concrete to 1 or 2 per cent of its normal value instead of to 50 per cent, as most of them do. It appears that the mere fact that the addition of a waterproofing agent increases the initial resistance by a considerable amount should not be construed as evidence of its value as a permanent protection against electrolysis.

Resistance of Granite and Limestone.—The specific measurements of granite and limestone were conducted on seven specimens, six of granite and one of limestone. Two kinds of granite were represented, gray and red. It was found that the resistance of limestone was from about six to eight times that of concrete, and that the resistance of granite was about one hundred times that of concrete. The latter value particularly is sufficient to warrant the use of granite in the footings and foundations of buildings to reduce the tendency of electric currents to flow between them and the earth.

Tests on Waterproofing Paints and Membranes for Concrete.—In a search for an effective and durable insulating coating which might be applied to the surface of concrete some tests were made of a number of waterproofing and damp-proofing paints and membranes. For the purpose of testing these paints and membranes they were applied to concrete cylinders like those used in testing for the effect of integral waterproofing, with the exception that the integrals were omitted in mixing the aggregate.

In applying the paints great care was exercised to obtain continuous and flawles films as far as the character of the paint would permit. The directions of the manufacturers were followed closely, and after th finishing coat had been applied the specimen were set aside for a period varying from fou to six weeks before testing. In order t avoid bruising the coating on the bottom o the cylinder where it came in contact wit the floor in handling the coating was ther treated with a layer of melted paraffin. Th prevented sticking, and as far as the tests wer conducted it seemed to have no detrimenta effect on the paints. Trial showed that ther is a vast difference between painting over paraffined surface and putting a layer o paraffin over dried paint. In the first instanc the paint never dries, but in the second no sign of deterioration ever appeared in the 40 or more paints tested in the course of this work.

The test on the paints was conducted in much the same way as the tests for the effect of integral waterproofings. The rate of absorption near the beginning of the test was taken as the true measure of the efficiency of the coating as a waterproofer because at any subsequent time the specimens had become more or less saturated with water with a consequent decrease in the rate of absorption, which was not due to any property of the coating. The greater portion of the coatings show conductivities at the end of seven days which are too large to allow them to be considered as insulating.

There are others, however, which show very favorable results in comparison with untreated normal specimens after being three months under test on 15 volts and their use where occasion demands it might be desirable if a life test on them should indicate that their insulating power is not too short lived. Life tests were not run in connection with the present work, partly because of lack of time.

Some tests were also run on specimens similar to the above treated with waterproofing membranes. These membranes consist of alternate applications of hot pitch, or asphalt, and fabric. They are designed for waterproofing against hydrostatic pressure, underground. In placing the membranes the concrete was first thoroughly swabbed with the melted compound, then a layer of fabric rolled on and it in turn swabbed with more of the melted compound. The operation was repeated until the required number of layers were obtained. Great care was exercised in breaking joints and filling all holes where leakage might possibly occur. The tops of the specimens did not come in contact with the water in testing so were not laid over with fabric but were simply painted with a thick coat of the melted compound.

The tests on the membranes were conducted in a manner slightly different from those on the paints. After the specimens were made up they were weighed, placed in a tank containing water 7 ins. deep and connected up with the embedded iron anode on the 15-volt circuit. The potential was left on continuously. At intervals of a month or two current readings were made and the specimens were removed from the tank and weighed. Two different brands of membranes were tested in this way, using from two to five layers. The tests were continued for 11 months and the electrical readings are those obtained at the end of that time. The effectiveness of these coatings as preventives of electrolysis in the manner for which they are adapted is unquestionable, provided, of course, that the membrane is applied with sufficient care.

Possibilities of Trouble From Electrolysis in Concrete Structures Under Practical Conditions.

In connection with the investigations described in the preceding section, the matter has been studied from the practical standpoint also, in order to determine, as far as possible, to what extent the conditions under which concrete can be injured in the laboratory may be expected to obtain in practice.

CONDITIONS NECESSARY FOR DAMAGE TO OCCUR.

A careful study of the data presented in the preceding pages shows conclusively that while there are conditions under which reinforced concrete may be seriously injured, such conditions are nevertheless exceptional rather than the rule. These exceptional conditions occur, however, with sufficient frequency to make the problem one of great importance, and fortunately most of these conditions are amenable to control. It has been seen that the most important essentials to the injury of concrete by electrolysis are moisture and a difference of potential between electrodes in contact with the mass of the concrete. At first thought it might appear that these two conditions are most common, since perfectly dry concrete, especially below grade, is seldom if ever found; while, as every electrical engineer knows, there are few places in our cities at the present time where appreciable differences of potential can not be found between any two points more than a few yards apart. The statement in regard to the rarity of dry concrete is made advisedly, since only the most minute quantities of moisture are necessary in order to impart to concrete a considerable conductivity. On the other hand, the concrete has to be made very wet in order to import to it a maximum of conductivity, and any reduction of the moisture content below the saturation point causes an increase in its resistance and a consequent decrease in the current which will flow through the concrete under a given potential gradient. As indicated by data already presented, the resistance of ordinary air-dried concrete, while extremely variable, is usually of the order of about ten times that of wet concrete, and for this reason concrete above grade is much less susceptible to electrolytic damage than if so located as to be permanently wet. It is not to be inferred, however, that air-dried concrete is immune from electrolysis troubles, but rarely would the voltage be high enough to produce trouble; and, in general, in the absence of special conditions to be mentioned in the next section, electrolytic damage to concrete above grade will be extremely rare.

The condition mentioned above, that the electric current must flow between electrodes in contact with the concrete, should be emphasized. The conduction being electrolytic, the reactions take place only at the electrodes, and in the absence of such electrodes no reactions occur within the concrete. The only effect, therefore, would be the slow removal of the water-soluble constituents, and hence the effect on the concrete would not be essentially different from that of slow water seepage.

SOURCES OF STRAY CURRENTS.

If there are electrodes embedded within the concrete, as in the case of reinforced concrete structures, the electrode effects described in the foregoing section may be expected provided the voltage is sufficient. The sources of potential differences in concrete structures may be classed under two heads, (1) those due to direct contact between the conductors of lighting or power circuits, and some part of the building and, (2) those which have their origin in stray currents from railways or other grounded power lines. The former may happen in any building containing electric wires, through defective insulation. It is not necessary, of course, that both sides of the line be grounded in the building itself, since if one side of the line is grounded on the building and the other grounded in some remote quarter of the system those portions of the building itself near the grounded wire will be subjected to a considerable difference of potential. If the wire be grounded directly on the concrete and not on the reinforcement, the comparatively small cross-section of the path of the current near the point of contact between the concrete and the wire will result in most of the total drop of potential to ground occurring within a restricted region near the wire, and it is only here that any damage may be expected, and since the current will be small the damage, if any, will be small. Ultimately, of course, all current that leaks off from the wire would pass out into the earth through the footings and foundations and through pipe systems entering the building. As a rule, the cross-section of these paths is so large in the aggregate that the potential gradients would not be sufficient to raise the temperature appreciably, and hence no appreciable damage is likely to occur. If the current be reversed, flowing to the building from outside, there would in time be some softening of the concrete in a thin layer under and around the steel structure terminating in the footings, but this would be under compression and not subjected to shear along the surface of contact between steel and concrete, so that failure here is extremely improbable. The only places where trouble is to be expected due to grounding of power wires directly on the concrete inside of a building is in the region close to the point of ground.

If, however, the power wire be grounded directly on a portion of the reinforcing material, the condition will be more serious, and the extent of the danger will be greater if there is a large quantity of the reinforcing material in metallic contact with the electric circuit. If this comprises a large part of the total reinforcement of the building, the condition might be serious irrespective of whether the positive or negative side of the line is grounded. If the ground is on the positive side, the potential gradient near the reinforcement may become high enough to cause rapid corrosion and consequent destruction of the reinforcing material. If, on the other hand, the reinforcing material is negative, there would develop a softened condition of the concrete near the surface of the iron which would practically destroy the bond, and this would probably be the more serious condition of the two, since the latter would not manifest itself by producing local cracks in the concrete, and might not become known until a large portion of the building has become weakened. However, while such a condition as this might occur, and if neglected become very serious, it is nevertheless a trouble that can be readily guarded against, as will be pointed out below.

The other source of current that might possibly give rise to trouble under certain circumstances is the ground return of railways. The current may enter a building in two different ways. First, if the foundations under the two opposite sides of the building are at different potentials, there would be a tendency for a certain amount of current to flow up through the foundation on the one side, through the walls and floors of the building and out through the foundation on the other side. This condition may be said to exist to a very small extent in practically all concrete buildings, but it is not one that need cause any alarm. In the course of numerous electrolysis surveys that we have carried out in various cities we have found that a potential difference exceeding a few volts due to stray currents between any two parts of a building is extremely rare, and this would almost inevitably be distributed over so great a distance that the potential gradient would not be sufficient to cause any appreciable trouble, in view of the experimental results set forth in the first section of this paper, which show that under ordinary conditions comparatively large potential gradients are required before any material damage is likely to occur.

The second way in which stray currents may enter a building is through water or gas pipes, lead cable sheaths and similar structures. In this case considerably larger differences of potential may be brought about between different portions of the building and between parts of the building and the earth. If the pipe systems come in contact only with the concrete and not with the reinforcing material, any damage that may occur will be slight and will be confined chiefly to the immediate vicinity of the pipes or cables, but if the pipes come into metallic contact with the reinforcing material, the latter comes to the same potential as the pipes and may become either anode or cathode, according to the condition of the pipes. Cases may arise in practice where differences of potential of serious magnitude may be produced in this way, some instances having been brought to our attention in which the reinforcing material was from 5 to 15 volts above or below the earth. Under most conditions damage would proceed very slowly under such differences of potential as these, but nevertheless wherever voltages of this magnitude exist it should be regarded as a dangerous condition and should be remedied at once.

INCREASED DANGER DUE TO PRESENCE OF SALT.

The above statements in regard to the liability of damage under low or moderate differences of potential are intended to apply only to concrete which contains no appreciable quantities of salt. The data given in the preceding section show that if a small quantity of sodium chloride or calcium chloride be added to the concrete the rate of deterioration proceeds many times faster, and under such circumstances much lower voltages should be regarded as dangerous. Even a small fraction of one per cent of chlorine in concrete is capable of increasing the electrolytic corrosion of the iron manyfold, and if salt has been added to the concrete during construction, or if it comes into contact with salt water afterwards, much greater precautions are necessary in order to prevent damage.

In the course of our investigations we have examined a considerable number of cases in which damage to concrete structures has been attributed to electric currents. Some of these have been reinforced structures and some have been without reinforcement. Among these we have not found any non-reinforced structures in which the conditions indicated that electric currents could in any way be responsible for the damage. Among the reinforced structures which have been called to our attention there are some in which electrolysis has been at least a contributing cause of the damage. We have not, however, seen any case in which serious damage has occurred in which there was not also present a considerable quantity of salt in the concrete, either from having been put there during construction or from contact with salt water in service. This is in accord with the results of our laboratory investigations, which show that under low or moderate voltages the rate of damage, in the absence of chlorine, is so slow as to be almost negligible, but that when salt is present rapid deterioration at both electrodes may be expected even on comparatively low voltage.

PROCEDURE IN MAKING VOLTAGE MEASUREMENTS ON CONCRETE STRUCTURES.

We desire to say a word in regard to the proper method of taking electrical measurements that are intended to show whether or not a structure is being affected by stray currents. It is not only important to read the voltages correctly, but it is equally important to take the measurements between the proper points and to properly interpret them. We have known cases where damage has been attributed to stray currents on the strength of potential measurements taken between the structure and the rails of a near-by steam railroad. The rails near the structure and for a half mile or so on either side were supported out of contact with the soil on wooden ties, so that the potential measurement that was being taken was between the structure and a point several thousand feet distant. It seems almost needless to say that a potential measurement of this kind is worse than worthless as an indication of the electrical condition of the structure under examination. If we are to get an idea of the extent to which currents may be flowing in a building by means of potential measurements, it is necessary that we take the potential readings between two points within or on the building itself and not between the building and some other structure more or less insulated from it. Similarly, if we wish to determine whether current is flowing into or out of a building, we should make the potential measurement between points on the building and thoroughly ground points near by. Failure to keep these facts in view appears to have been responsible for a good many erroneous reports of damage to concrete structures by stray currents.

Protective Measures to Prevent Electrolysis.

In the following discussion of means of reducing or preventing electrolytic troubles it is to be understood that the measures recommended are to be regarded as necessary only in those cases in which there is reasonable probability that the electrical conditions may become dangerous. In considering measures that may be taken to prevent damage or to cure electrolytic troubles it is not necessary to consider structures which have no metal imbedded in them, since, as pointed out above, these are immune from electrolytic effects. As regards reinforced-concrete buildings (or concrete buildings with metal conduits imbedded in the concrete) there are two cases to consider, new structures in process of building and structures already completed.

EXCLUSION OF SALT.

In the case of new structures of reinforced concrete salt should be omitted altogether if there is the slightest probability that they may ever be subjected to the action of electric currents. Calcium chloride is quite as bad as sodium chloride, it being the chlorine that does the harm. Since the addition of even a fraction of 1 per cent of chlorine is sufficient to increase the rate of damage a hundredfold, it is impossible to use a sufficient quantity of salt to lower the freezing point of concrete perceptibly without making the structure decidedly more vulnerable to the action of electric currents.

WATERPROOFING BELOW GRADE.

A good deal can be accomplished also by proper construction of the basement, foundations, and footings of the building. If beneath the foundations and outside the basement walls a layer of insulating material be placed, it will prevent the access of electric currents through the building foundations. While such insulating materials can readily be made that will give permanent increase of resistance, we have yet to see one that will, in the presence of water, remain completely insulating over a period of years. We have tested a great many waterproofing membranes, but in the presence of water all have sooner or later begun to acquire considerable conductivity and show gradual deterioration. It is, of course, not necessary that such insulation should be perfect in order to be of value, since any considerable increase in the resistance which it may produce in the path of current flow will be useful. For this reason a waterproofing medium, such, for instance, as one made up of multiple layers of fabric treated with pitch, will have sufficient insulating value to be quite effective in preventing the passage of electric currents, and would probably be of considerable value even after the waterproofing has ceased to be perfect.

ADDITION OF WATERPROOFING COMPOUNDS.

Attempts to increase the resistance of concrete by the addition of so-called waterproofing compounds to the cement before mixing have uniformly failed to give satisfactory results. As pointed out in the first section of this report, none of these compounds or mixtures that have been brought to our attention are of great value in this connection, and reliance should not be placed on them as a means of protecting structures against electrolysis.

CONSTRUCTION OF FOUNDATIONS.

A good deal can be done in the way of increasing the resistance between the building and ground by a proper selection of materials for the foundations. Blocks of granite, frequently used in such work, have a much higher resistance than concrete, a number of specimens tested by us showing, in the water-soaked condition, a resistance approximately one hundred times as great as that of wet concrete, and hence if granite blocks be interposed between the footings and the soil the tendency of the building to pick up stray currents will be greatly reduced. We regard this matter in most cases as of secondary importance if the potential differences in the region of the building are as small as usual, since any currents which a building may pick in this way will be too small to do any appreciable harm. In some cases, however, the use of granite footings may be justified as a measure of precaution.

The foregoing statements are not applicable to structures already built, but the following preventive measures may as a rule be applied to old as well as new structures.

ELECTRIC WIRING.

In wiring a concrete building with direct-current circuits it is of the greatest importance that the construction be such as to preclude the possibility of either side of the circuit coming in contact with the concrete. The insulation should be of the best grade and the wires should be inclosed in continuous metal conduit. When practicable periodic tests should be made on the insulation and any defect remedied. This can usually be done in the case of large structures, but in small structures generally it can not be considered practicable. If the power supply is a private plant belonging to the building the installation of ground detectors is desirable. If these precautions are taken it will not be necessary to insulate the conduits from the building as has been sometimes proposed.

INSULATION OF PIPE AND CABLES.

By far the most important path of entry of electric currents into concrete structures from outside sources is through the pipes and lead cable sheaths which enter the building, and it is only through these paths, as a rule, that sufficiently high potential differences can be produced to result in damage to the structure. The most effective remedy, therefore, and at the same time a comparatively simple one, is to introduce insulating joints into the pipes before they enter the building. Such joints can easily be made, and are in fact now used in many places, although not so far as we are aware in this particular connection. The only condition to guard against is the possibility of developing too high a difference of potential across the joint which might give rise to electrolytic injury to the pipes; but this can be obviated by proper construction of the joint, as by giving it a long leakage path and by putting several insulating joints in series.

When a pipe line passes through a building it will be necessary, if insulating joints are used at all, to use them on both sides of the building. To use them on one side only would result in making the building more strongly positive or negative to the earth than it would be without such joints and thus the danger would be increased. If the insulating joints are put on both sides, however, there will be no likelihood of currents entering the structure to any appreciable extent, except in special cases where the difference of potential between the broken sections of pipe becomes very high, say of the order of 10 or 15 volts or more. When this condition occurs, however, it can be alleviated by shunting around the insulated section of pipe within the building by means of a copper cable, preferably an insulated one.

It was stated above that lead-covered cables also form a possible source of entry of stray currents into buildings. It is, of course, possible to break up the continuity of lead cable sheaths by inserting insulating joints, but there are more serious objections to doing this than in the case of water or gas pipes. Nevertheless in some instances this is to be recommended; but it would be usually better simply to insulate the cable from the building, since only a very low degree of insulation would be necessary, such as would be obtained by carrying the cable on wooden supports and keeping it out of actual contact with the concrete and reinforcing material.

MAKING THE REINFORCING MATERIAL NEGATIVE.

A method of protecting reinforced concrete structures that has frequently been proposed, viz., by making the reinforcing material continuous throughout the building and connecting it to the negative terminal of a low-voltage generator, should be referred to here, but only to point out the danger attending it. While there is no question but that corrosion of the reinforcing material might be prevented in this way, the experiments described in the first section of this paper show that under ordinary low voltages there is greater danger in having the iron negative than positive on account of the destruction of the bond in the former case. Any condition, therefore, which may cause the reinforcing material to become negative to the concrete should be carefully avoided.

IMPROVING THE NEGATIVE RETURN OF RAILWAYS.

In discussing means of preventing damage to buildings due to stray currents, we should not lose sight of the fact that, after all, the most effective way of reducing damage due to stray currents is by removing, as far as practicable, the cause of such currents. This means a proper design, construction and maintenance of the railway distribution system, whereby potential gradients in the earth are not permitted to reach such high values as to give rise to serious trouble. The protective measures against stray currents mentioned above are rather to be regarded as precautionary ones, rendered necessary only on the failure of the railway companies to provide adequate means for returning their currents to their source. If the negative return systems of the electric railways of this country were uniformly as good as those maintained in the principal countries abroad, the problem of damage to buildings by stray currents would practically disappear.

GROUNDING OF METALLIC CONDUITS.

The practice at present followed in many instances, and made compulsory in some city ordinances, of grounding all metallic conduits in contact with the concrete, is not to be recommended as a general rule. There are, of course, many cases under which such grounding would not bring about a condition dangerous to the concrete, but in some cases at least the results of such grounding might be serious.

The above recommendations against the grounding of conduits is based on several considerations. If the ground were made on water pipes without insulating joints to prevent flow of current between the pipe system and the building as is commonly done, and if the district were one in which the pipes were at a higher or lower potential than the earth, the conduits would be made either anode or cathode, a condition which in general should be regarded as unsafe, and this condition would in fact be worse than the unrestricted entry of pipe systems into the buildings, which should be guarded against as pointed out above.

If, on the other hand, the ground is made through a ground plate under or near the building this would bring all of the conduit system to the same potential as the ground plate, and this being of small area compared to that covered by the building the footings of the building remote from the ground plate may show a considerable difference of potential against the ground plate and consequently against the conduit of the building, thus giving rise to flow of current between the conduit and the structure itself. That this may be the case is quite evident from the fact that voltage surveys made in several cities in the course of this investigation show that in a great many instances large potential differences are found between two points in the earth that might easily fall within the space covered by a large building. This is particularly true if the measurements are taken over distances at right angles to the electric railway tracks and very close thereto in either the positive or negative areas. Places were found quite frequently where a difference of potential of several volts would be noted between points not over 100 ft. apart. If a building were placed in such a locality, the different footings and different parts of the foundation would be at a considerable difference of potential with respect to each other and this would give rise to a tendency for the current to flow in at some of the footings and out at others. The resistance opposing this current flow would be mainly that of the footings themselves and the earth immediately beneath and such resistance would be encountered at both the entrance and exit of the current. If, however, ground plates be placed under the building at one locality and connected to the metal structure of the building the resistance in the current path would be reduced and larger currents would flow through the building producing either anode or cathode effects in the footings remote from the ground plate, according to the direction of current flow. While such voltages as would be likely to occur here would probably not give rise to any considerable trouble from anode effects, the cathode effects might be serious not only here but elsewhere in the building. The point to be emphasized is that a ground at one point does not assure relief from differences of potential, because of possible high local potential gradients in the earth. Of course, if the vertical walls and columns of the building were of non-reinforced concrete, and fairly dry, then the resistance would no doubt be sufficient to guard against trouble from this cause, but with any considerable amount of iron in the walls, either as reinforcement, piping, etc., such would not be the case and a local ground would have a tendency to increase the danger from this source. A ground covering the entire basement of the building, or a separate plate under each footing and under the foundation walls, all interconnected, would of course relieve this danger, but the desirability of attempting such extensive grounding is questionable, partly because of the cost and the lack of permanence of such grounds and their connection, and partly because grounding of this character would tend to increase danger from other sources, as described below.

Another argument sometimes advanced in favor of grounding the metal work of a building, including the electrical conduits, to a ground plate is that in case one of the power wires comes in contact with the conduit the grounding of the conduit would prevent any difference of potential between the building and the earth. An extensive study of electrical grounds in general, which was made incidental to this work shows that such is not the case. In the first place, it is practically impossible to make a ground through a metal plate that will have a sufficiently low resistance to prevent such differences of potential from arising due to a contact between wires and conduit. Tests conducted on grounds made at the Bureau of Standards, and also of those commercially used on low voltage secondaries and other places, show that these grounds are invariably of comparatively high resistance. The best grounds tested have been those made at the bureau and the results of these tests are instructive. These were made with plates 8x12 ft. laid in wet ground with a bed of crushed coke below the plates and a layer of the same above, the whole being heavily salted and thoroughly flooded. In wet weather, when the ground is everywhere saturated with water, the resistance of these grounds is about 15 ohms, as shown by measurements against each other and against the water-pipe system of the city of Washington. During a dry period the resistance is much higher. Consider, now, that the metal work of a building, including the electrical conduit, is connected to such a ground and that the wires become grounded on the conduit. The current can go to ground both by way of the ground plates and also through the footings and foundations, the two paths being in parallel and independent of each other. If the circuit be 110 volts, several amperes may flow through the ground plate, but this, under usual conditions, would not cause any disturbance in the system and would probably go unnoticed except in a very small building using but little power. This parallel circuit through the ground plate would have no more effect on the potential of other parts of the building than the throwing of a load of several amperes in parallel with a lamp on an ordinary lighting circuit would have on the potential of the lamp. It would not, therefore, have any tendency to protect the structure from damage due to contact between a power wire and its conduit. On the other hand, it is extremely difficult to insure that all parts of the conduit shall be at all times in perfect electrical connection with the other metal parts of the building, because considerable resistance may be developed at joints, so that the portion of the conduit in contact with the wire may be but imperfectly connected to the ground plate. If at the same time the reinforcing material be connected to a ground plate there would be a great tendency for current to flow from conduit to reinforcing material or the reverse. Owing to the close proximity of conduit and reinforcement in many places, a comparatively low resistance joint sufficeint to take up 2 or 3 per cent only of the voltage of the line (which would require but a fraction of an ohm in the case being considered, viz., a 15-ohm resistance in the ground plate) would give rise to a potential difference of several volts between conduit and reinforcement, which might do great harm, especially if the reinforcement is cathode.

While, of course, it would be possible, by going to considerable expense to make a ground of lower resistance than those just considered, it does not seem that it would be practicable to make and maintain permanently a sufficiently low resistance to give the protection sought.

It is desirable, however, to connect the metal work of the building together as far as practicable, provided such metal work is not in turn connected directly to pipe lines and cables entering the building. In other words, it is recommended that conduits, and such pipes as are separated from their mains by means of insulating joints, be interconnected electrically, but that this group of interconnected metal work should not be grounded directly to earth for the reason given above. In addition, it may be said that if the low-voltage side of alternating-current circuits be grounded at the neutral point there is no material advantage to be gained by grounding the conduit.

Conclusions.

The following conclusions are drawn from the investigations:

(1) The observations of previous investigators that the passage of current from an iron anode into normal wet concrete caused the destruction of the test specimen by cracking the concrete were only partly confirmed. This effect was found not to occur in most of the specimens tested when the potential gradient was less than about 15 volts through a distance of 3 ins., or about 60 volts per foot. These figures must be considered as but roughly approximate as they depend much on conditions.

(2) Of the numerous theories that have been advanced for the cracking of reinforced concrete due to electric current the one which attributes it to oxidation of the iron anode following electrolytic corrosion has been fully established. The oxides formed occupy 2.2 times as great a volume as the original iron, and the pressure resulting from this increase of volume causes the block to crack open.

(3) Metals which do not form insoluble end products of corrosion and all non-corrodable anodes never cause cracking of the concrete as a result of the passage of an electric current.

(4) The mechanical pressure developed at the iron anode surface by corrosion of the iron has been measured in a number of cases and has been found to reach values as high as 4,700 lbs. per square inch, a value more than sufficient to account for the phenomena of cracking that have been observed.

(5) Suggestions of some engineers that copper-clad steel or aluminum be used as reinforcing material have been shown to be impracticable, since the copper coating is readily destroyed and the aluminum is attacked by the alkali in the concrete.

(6) Corrosion of iron anodes even in wet concrete is very slight at temperatures below about 45° C. (113° F.)

(7) For any fixed temperature the amount of corrosion for a given number of ampere-hours is independent of the current strength.

(8) The lack of corrosion of the iron at temperatures below 45° C. is due to the inhibiting effect of the $Ca(OH)_2$ and possibly other alkalies in the concrete.

(9) The rapid destruction of anode specimens of moist concrete at high voltages (60 to 100 volts or more) is made possible mainly by the heating effect of the current, which raises the temperature above the limit mentioned above. If the specimen be artifically

cooled no appreciable corrosion occurs, and no cracking results.

(10) The potential gradient necessary to produce a temperature rise to 45° C. with consequent corrosion, in the specimens used, was about 60 volts per foot. For air-dried concrete it is much higher. This shows that under actual conditions corrosion from stray currents may be expected only under special or extreme conditions as noted below. These figures are but roughly approximate since they will vary greatly with the conditions, such as the size, form and composition of the specimen, but they serve to show the order of magnitude of the voltage required to produce trouble.

(11) Since the passivity of iron in concrete is due chiefly to the $Ca(OH)_2$ present it appears probable that old structures in which the $Ca(OH)_2$ has been largely converted into carbonate will be more susceptible to the effects of electric currents than comparatively new concrete with which the foregoing experiments have been made. The increase in the efficiency of corrosion would, however, be at least partly offset by the increase in the resistance of the concrete which would accompany the change.

(12) The addition of a small amount of salt (a fraction of 1 per cent) to concrete (as is frequently done to prevent freezing while setting) has a twofold effect, viz., it greatly increases the initial conductivity of the wet concrete, thus allowing more current to flow, and it also destroys the passive condition of the iron at ordinary temperatures, thus multiplying by many hundreds of times the rate of corrosion and consequent tendency of the concrete to crack. Salt should, therefore, never be used in structures that may be subjected to electrolytic action. Further, reinforced concrete structures built in contact with sea water, or in salt marshes, are more susceptible to electrolysis troubles than concrete not subjected to such influences.

(13) Specimens of normal wet concrete carrying currents increase their resistance a hundredfold or more in the course of a few weeks, which fact still further lessens danger of trouble.

(14) The rise of electrical resistance is probably due to a number of causes, among which are the precipitation of $CaCO_3$ within the pores of the concrete, thus plugging them up. A slight amount of salt tends to prevent this precipitation and interferes with the rise of resistance, thus still further emphasizing the detrimental effect of salt.

(15) Contrary to the observations of previous investigators there was a distinct softening of the concrete near the cathode. This begins at the cathode surface and slowly spreads outward, in some cases as far as ¼ in. or more. After exposure to the air this softened layer becomes very hard again, but remains brittle and friable.

(16) The softening effect at the cathode noted above, caused under the conditions of the experiments, practically complete destruction of the bond between reinforcing material and the concrete, reducing it to a few per cent of its normal value.

(17) Unlike the anode effect which becomes serious in normal concrete only on comparatively high voltages, the cathode effect develops at all voltages used in the experiments, the rate being roughly proportional to the voltage in a given specimen.

(18) In general the cathode effect occurs under conditions which may not infrequently occur in practice and is therefore probably a more serious matter practically than the anode effect about which so much has been written. This trouble is unlikely to be serious, however, except where the concrete is wet and the potential differences rather large.

(19) The softening of the concrete at the cathode is due chiefly to the gradual concentration of sodium and potassium near the cathode by the passage of electric current. In time the alkali becomes so strong as to attack the cement.

(20) Softening at the cathode is increased by increasing the sodium and potassium content of the cement, and reduced by diminishing this content, at least within the range below 10 per cent of the total salts.

(21) The softening of the concrete has never been observed, except very close to the cathode, the main body of the concrete remaining perfectly sound. Numerous tests show conclusively that the crushing strength of the main body of the concrete is not reduced even when the potential gradient is maintained at 175 volts per foot for over a year.

(22) Because of the cathode effect noted above, the proposal to protect reinforced concrete buildings by maintaining the reinforcing material cathode as by a battery or booster would be much more dangerous than no protection at all.

(23) Aside from slight heating, which is usually negligible, the only effect which an electric current has on unreinforced concrete is to cause a migration of the water soluble elements. Consequently, in the absence of electrodes, the ultimate effect of current flow on the physical properties of the concrete is not materially different from that of slow seepage, which also removes the water soluble elements. Non-reinforced concrete buildings are therefore immune from trouble due to stray earth currents. They might, however, be injured by the grounding of power wires within the structure since these or the inclosing conduits would then act as electrodes.

(24) Conditions arise in practice which give rise to damage due to stray currents, but the danger from this source has been greatly overestimated. While precautions are necessary under certain conditions, there is no cause for serious alarm.

(25) If reinforced concrete could be thoroughly waterproofed, it would greatly increase its resistance and diminish accordingly the danger from either the anode or cathode effects. It should be emphasized, however, that waterproofing to prevent electrolysis is a much more difficult matter than waterproofing to maintain a moderate degree of dryness, because of the much higher degree of waterproofing required in the former case. It has been found that practically all of the waterproofing agents now on the market that are intended to be mixed with the concrete, are of little value as preventives of electrolysis. Waterproofing membranes, etc., applied to the surface can be made more effective and when properly applied may have considerable effect in preventing the entry of earth currents in the concrete.

(26) Painting or otherwise coating iron with an alkali resisting metal preservative before embedding it in concrete may serve to minimize the dangers of electrolysis, but no such coating has been found that does not prevent the proper formation of the bond between the concrete and iron when the concrete sets.

(27) In order to insure safety of reinforced concrete from electrolysis the investigation shows that potential gradients must be kept much lower in structures exposed to the action of salt waters, picking baths, and all solutions which tend to destroy the passive state of iron.

(28) All direct current electric power circuits within the concrete building should be kept free from grounds. If the power supply comes from a central station the local circuits should be periodically disconnected and tested for grounds and incipient defects in the insulation. In the case of isolated plants ground detectors should be installed and the system kept free from grounds at all times.

(29) All pipe lines entering concrete buildings should, if possible, be provided with insulating joints outside the building. If a pipe line passes through a building and continues beyond, one or more insulating joints should be placed on each side of the building. If the potential drop around the isolated section is large, say, 8 or 10 volts or more, the isolated portion should be shunted by means of a copper cable.

(30) Lead-covered cables entering such buildings should be isolated from the concrete. Wooden or other non-metallic supports which prevent actual contact between the cable and the concrete will give sufficient isolation for this purpose. Such isolation of the lead-covered cable is desirable for the protection of the cable as well as the building.

(31) The interconnection of all metal work within a building is an advantage where practicable, provided that all pipe lines entering the building are equipped with insulating joints and lead cables are taken care of as indicated in the preceding paragraph, but the grounding of such interconnected metal work or any part of it to ground plates or to pipe lines outside of the insulating joints is to be strictly avoided.

(32) In making a diagnosis of the cause of damage in any particular case, the fact that a fairly large voltage reading may be obtained somewhere about the structure should not be taken as sufficient evidence that the trouble is due to electrolysis. The distance between the points, and particularly the character of the intervening medium, are of much greater importance than the mere magnitude of the voltage reading. As a precautionary measure, however, all potential readings about a reinforced concrete structure should be kept as low as practicable.

Passenger Elevators for Continental & Commercial National Bank Building, Chicago.—The 23 passenger elevators in the Continental & Commercial Bank Building, Chicago, are known as the gearless traction electric type, and are equipped with American wire rope. This type possesses some features which differ considerably from those of the geared type; for example, the hoisting cables are not wound on a drum, but drive over a sheave which is bolted directly to the armature spider of the motor by means of a flange, integral with the spider. The necessary tension in the wire cables to secure tractive force between the driving sheave and the hoisting cable is secured by passing the cables partially around the traction driving sheave and then around an idler leading sheave, forming a complete loop around both sheaves. The car and the counterweight are suspended from opposite ends of the cable, thus avoiding the use of intermediate gearing. The motors used in this elevator system are of the slow-speed shunt-wound type, with a capacity of 35 HP. The normal capacity of each elevator is 2,500 lbs., lifted at a speed of 500 ft. per minute. To insure safety the motor is equipped with a governor which regulates the motor speed regardless of the load. At the bottom of each shaft there is placed an oil-cushion buffer, which brings the car to a stop, being regulated by the escape of oil from one chamber of the buffer to another.

American Railway Engineering Association.—The fifteenth annual convention of this Asociation will be held at the Congress Hotel, Chicago, March 17 to 20. In addition to the regular addresses and reports of committees and the annual election and dinner, the program includes a visit to the exhibit of the National Railway Appliances Association in the Coliseum and Armory.

Improvement of Roads in British Columbia.—A recent statement of the minister of public works of British Columbia shows that \$1,000,000 has been expended on roads in the province during the past season. In this was included \$900,000 for bridges, \$70,000 for road location work, \$100,000 on machinery, and \$100,000 for maintenance during the winter months.

About \$100,000 was spent on the Banff-Windemere road, built in connection with the Canadian Pacific Railway, along the Columbia River Valley. A considerable sum has been expended in the connecting of existing roads and on their standardization.

The more important bridges have been two steel structures, erected over the Columbia at Trail at a cost of \$140,000 and over the Thompson at Lytton at a cost of \$60,000, while a contract has been recently let for constructing a bridge at Takhum, near Nelson.

Water Works

Design, Construction and Operation of the New Water Purification and Pumping Plants at Bridgeton, N. J.

The new 3,000,000 gal. water filtration and umping plants at Bridgeton, N. J., a city of approximately 15,000 inhabitants, were placed n regular operation on Nov. 1, 1913. The present article describes the design and contruction of these works and gives the operaion results for the months of November and December, 1913. The supply is drawn from he West Branch of the Cohansey River, a tream having a dry weather flow of about 3,000,000 gals. per day, and receiving the runoff from a watershed of approximately 46 quare miles in area. This drainage area is parsely inhabited, containing no towns, and omprising principally cultivated farm land and undrained swamps. The raw water shows igns of sewage pollution, on analysis, but is of fair quality for a water supply. It is seldom turbid but always has a high color derived from organic matter in the swamps. The vegetable matter in the swamps also gives the vater a distinct vegetable or earthy odor, more noticeable in the fall of the year than at other seasons. Neither the filters nor will ny chemical treatment entirely remove this undesirable quality during September and October when the odor is the strongest in the raw water. During the rest of the year it is not noticeable to the ordinary consumer. The vorks here described were designed and the construction was superintended by Mr. Henry Ryon, commissioner of public works at Bridgeon while assistant to Clyde Potts, consulting engineer, New York City. The information here given is taken from an article prepared by Mr. Ryon and published in a recent issue of The Cornell Civil Engineer. After the completion of the plants Mr. Ryon was employed by the city to operate the works for a year and to instruct the various employes in their duties.

INTAKE.

Intake House.—It was the original intention to take the raw water from the lower end of a pond a little over a mile in length, but since he pond was used to some extent for boating, popular opinion forced the city to run an inake line to the upper end of the lake, even hough it was found that traversing the lake cleared the water of sediment and would have made its purification easier. The intake house, brick on a concrete foundation, was accordngly located on the side of the stream at the head of the lake. The water enters the house hrough an opening in the foundation wall 3 t. high by 6 ft. wide and thence passes through he house and into the conduit leading to the purification plant.

Intake Screens.—To prevent the entrance of ish and floating material into the intake conduit, three movable screens have been placed n the intake house. The first covers the openng mentioned previously and is made of ½ n.x2 in. iron bars placed 1 in. apart. Back of his screen are two screens of brass wire, the irst of ¾-in. mesh and the second of ⅜-in. mesh. All screens are arranged to be easily and separately raised for cleaning. A wooden shut-off gate is also provided in this intake for shutting off the water.

Raw Water Conduit.—From the intake house a conduit of 30-in. vitrified tile pipe, approximately 8,500 ft. long, passes under the lake and hrough the bank on the west side of the raceway, 14 ft. deep in places and carries the water to the coagulation basin of the filter plant. This conduit, with the gates open wide, will arry 4,200,000 gals. per 24 hours. The difference in level between the water at the intake and that in the coagulation basin under working conditions is 4 ft.

In general, the line was laid in the gravel without any foundation, although at points it was found necessary to support the tile with concrete. The joints were made of 1:1 cement mortar forced into position from the inside of the pipe. Under working conditions the pipe is required to stand an internal pressure of from 0 to 14 ft., and as each section of 400 ft. was completed it was tested with this pressure before the trench was back-filled. No appreciable leaks were discovered. The line cost $3.70 per lineal foot complete.

PURIFICATION PLANT.

Coagulation Basin.—The lower end of the intake line connects to the coagulation basin. This basin is of reinforced concrete, rectangular in form and has a capacity of about 150,000 gals. for about one hour's flow. The walls are 15 ins. thick at the bottom and 12 ins. thick at the top, being dependent for their stability entirely on the concrete tie beams across the top, and the floor which holds the bottoms of the walls together.

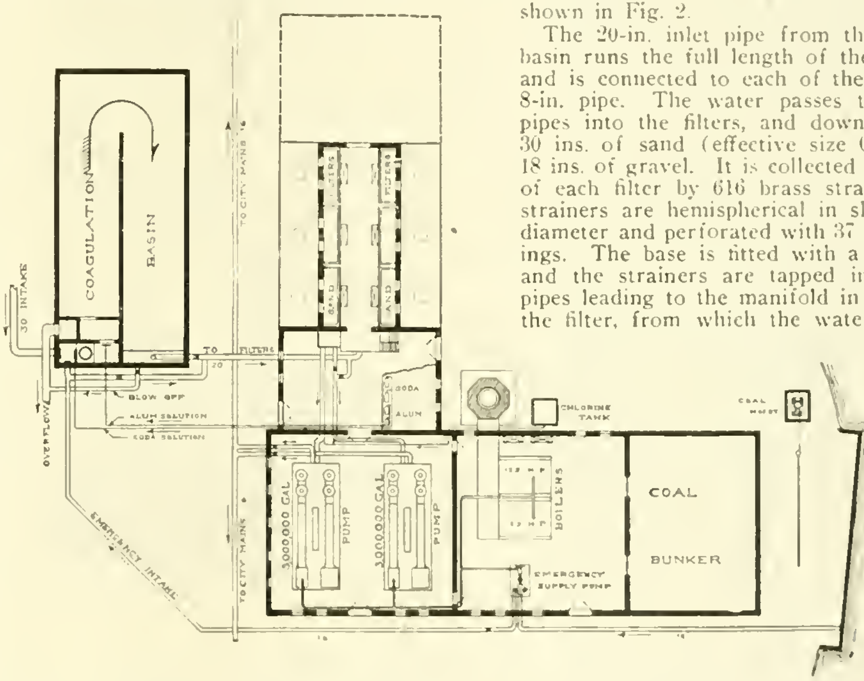


Fig. 1. General Plan of Bridgeton, N. J., Water Works Pumping Station and Purification Plant.

The water enters the bottom of an inlet well about 1 ft. square, at the southwest corner of the basin, rises in this chamber and passes through a 30-in. balanced float valve into the basin proper. This float valve automatically cuts off the inflow when the water in the basin reaches the level for which the valve is set. An over-flow weir in the north side of the inlet chamber at the level of the water at the intake house provides for any surge due to the sudden closing of the valve, and prevents damage being done to the vitrified tile conduit due to water hammer. A 12-in. blow off gate is provided for drawing off the upper portion of the water in the basin. The lower 4 ft. is, however, too low to drain to the river by gravity, and a pipe from the bottom of the basin is connected to the sludge well in the main building from which the water may be pumped by a 6-in. centrifugal pump operated by a 10 HP. steam engine.

The soda and alum used in treating the water are applied in the order named at the entrance of the coagulation basin. The water moves north on the west side of the middle baffle and south on the east side to the concrete collecting trough from which a 20-in. pipe carries it to the sand filters. To provide means of operating the plant while the basin is being cleaned a by pass has been built across the end of the basin with suitable valves as shown in Fig. 1.

Besides being fed from the main intake, the basin can, in case of emergency, be supplied direct from the river beside the plant by means of the 12-in. Worthington centrifugal pump located in the boiler room. This pump has a capacity of 4,000,000 gals. per day and is direct connected on the same base to a 25 HP. Blake engine.

Filters.—The filters, each 12 ft. by 14 ft. 6 ins., are arranged in two rows of three each on opposite sides of the operating floor and pipe gallery. The construction is entirely of concrete. The floor consists of a 6-in. slab supported on longitudinal beams which in turn are carried by the heavy beams extending from the center posts to the outside walls and act not only as beams but as division walls between the filters. The general arrangement is shown in Fig. 2.

The 20-in. inlet pipe from the coagulation basin runs the full length of the filter house, and is connected to each of the filters by an 8-in. pipe. The water passes through these pipes into the filters, and down through the 30 ins. of sand (effective size 0.4 mm.) and 18 ins. of gravel. It is collected at the bottom of each filter by 616 brass strainers. These strainers are hemispherical in shape, 1 in. in diameter and perforated with 37 1/16-in. openings. The base is fitted with a ⅜-in. nipple, and the strainers are tapped into 2-in. iron pipes leading to the manifold in the center of the filter, from which the water passes to a rate controler of the Venturi meter type and drops into the clear water basin beneath.

The operation of this type of controler is dependent on the fact that the difference in pressure at the throat and full section of a Venturi tube varies with the quantity of water passing through it. The apparatus consists essentially of a balanced valve through which the water from the filter must pass. The stem of this valve is attached to a rubber diaphragm and two pipes, one from the throat and one from the full section of the Venturi tube, are connected on opposite sides of the diaphragms. Any variation in the quantity of the flow, with the consequent variation in pressure at the throat and full of the tube raises or lowers the diaphragm and opens or closes the balanced valve, keeping the flow of water uniform, and preventing an excessive flow when the filter is first started.

Each filter is provided with a system of air pipes by means of which air may be forced through the sand during the washing. The main air pipe is a 4-in. wrought iron pipe and the small laterals ⅜-in. brass pipe placed 6 ins. on centers and each perforated on the under side with 15 1/16-in. openings. The air is supplied at 5 lbs. pressure by a No. 3 Roots blower direct connected to a 25 HP. Troy engine. The blower is usually run at 200 R. P. M. delivering 700 cu. ft. of air per minute. (4 cu. ft. per square foot of surface.)

Filter Operation.—All valves for each filter are controlled hydraulically from an operating table of the usual type. Mercury U-tube gages are provided on each table to indicate the loss of head through the filters. These gages are very similar to the loss of head gages commonly used in hydraulic work. The pipe from one end of the U is connected to the inlet, and that from the other to the outlet. An aspirator is provided on each table for drawing water into the connecting pipes and filling the gage. These aspirators also serve for obtaining samples of filtered water, drawing the water into a glass globe on each table from which the sample is taken by means of a faucet.

As stated above the coagulant is applied to the water as it enters the coagulation basin. The chemical solutions are mixed in concrete tanks on the upper story of the building. These tanks are in duplicate. Each of the two alum tanks has a capacity of 3,500 gals. and each of the two soda tanks a capacity of 1,950 gals. The tanks are fitted with hot and cold water and perforated brass air pipes are provided on the bottoms of each tank for agitating the solutions. From the mixing tanks the solutions flow to the porcelain lined orifice tanks in the laboratory. The level in these orifice tanks is kept constant by float valves, and the outlets, brass needle valves, are adjusted to feed the amount of chemical necessary.

To supplement the filtration process and effect a saving in the amount of alum required, arrangements have been made to apply chlorine to the filtered water whenever it is deemed necessary. The "chloride of lime" solution used for this purpose is mixed in a 6 ft. by 6 ft. square concrete tank having a capacity of 1,000 gals. This tank is located outside of the building in order to prevent the fumes of chlorine from coming in contact with the machinery. One inch lead pipes lead from this tank to the sections of each of the two pumps. The partial vacuum produced in the suctions draws the solution into the water as it comes from the clear water basin and the passage through the pumps thoroughly mixes the chlorine solution into the water. Brass needle valves are used to regulate the quantity applied.

In the application of both the coagulant and the hypochlorite solution it was found difficult to keep the needle valves regulated, and more difficult to induce the attendants to give them proper attention. Floats were, therefore, placed in each solution tank and wires run to a large gage board in the engine room. As the solutions in the tanks flow out, the float falls and the weight suspended on the gage board rises. The content of 1 in. depth of the tank being known, the number of inches that the weight rises per hour shows the quantity of chemical being applied. This method of control proved far more satisfactory than calibrating the orifices.

The filters are operated at the rate of 500,000 gals. per day (120,000,000 gals. per acre per day.) Immediately after washing, the loss of head through the filters is about 2½ ft., and gradually increases to about 6 ft. when it becomes necessary to wash the filter on account of the diminished quantity of the effluent. The time between washings varies with the condition of the water and is from six to ten hours. In washing the air is used three minutes and the water (from the city mains) on the average approximately seven minutes. After the filter is started the effluent is wasted for 15 minutes, it being found by experiment that this time is required to bring the filter to proper efficiency.

The quantity of alum required under ordinary conditions is about 0.5 grain per gallon, but after a heavy rain it is necessary to use as high as 1.5 grains to give satisfactory results. Soda is used to partially neutralize the alum, for the average alkalinity of the raw water is only 7.0. This alkalinity has caused some difficulty when the character of the raw water has required the application of the larger quantities of coagulant, for it has been found here, as at some other places, that when soda is added in sufficient quantity to neutralize more than half the alum the floc (aluminum hydroxide) will not form properly. Arrangements are therefore being made to apply lime instead of soda when the alkalinity is very low, and the turbidity and color high.

The chloride of lime, applied as mentioned above, is used on the average at a rate of 0.20 parts of available chlorine per million. It might be mentioned, that the actual amount of chloride of lime used is 10 lbs. per 1,000,000 gals., which should give 0.40 p. p. m. Only about half this amount, however, can be accounted for by chemical examination of the solution as it enters the water. A higher efficiency could probably be obtained if some device were installed in the solution tank to agitate its contents.

TABLE I.—AVERAGE RESULTS OF DAILY PHYSICAL AND BACTERIOLOGICAL TESTS.

	November			December		
	Raw.	Filtered.	Tap.	Raw.	Filtered.	Tap.
Color	56.0	8.0	.....	57.0	1.0	....
Turbidity	2.0	0.0	.....	1.0	0.0	.....
Bacteria	98.9	11.7	5.8	67.3	9.2	2.1
B. Coli in 0.1 cc	37%	0.0	0.0	15%	0.0	0.0
B. Coli in 1.0 cc	100%	0.0	0.0	90%	0.0	0.0
B. Coli in 10.0 cc	....	33%	0.0	....	23%	0.0
Alkalinity	7.0	5.2	.....	7.1	5.3	....
Alum used	0.61 gr. per gal.			0.60 gr. per gal.		
Soda used	0.12 gr. per gal.			0.13 gr. per gal.		
"Available chlorine"	0.23 parts per mil.			0.16 parts per mil.		

Laboratory Procedure.—To provide for the intelligent operation of the plant, a small well equipped laboratory has been provided. Here, since the plant was first put in operation, daily examination of the raw and filtered water has been made, complete chemical analyses of the water are made monthly by the State Board of Health, and for that reason the laboratory work has been confined to such daily tests as are necessary to show the condition of the water and efficiency of the plant.

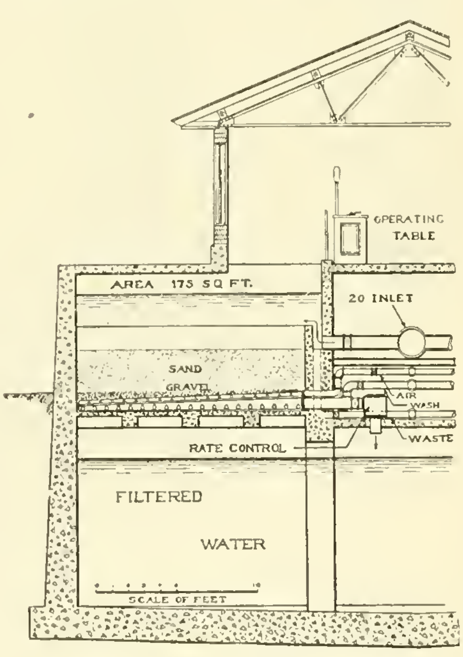


Fig. 2. Half Cross Section Through Bridgeton Filters.

Turbidity and color are measured in the raw and filtered water by comparing with standard solutions, and indicate immediately although approximately the efficiency of the filters. High color and turbidity always occur at the same time as high bacterial counts, and the results of these tests form a convenient basis for regulating the quantity of coagulant. The turbidity of the raw water has never yet been above 25, is readily removed and in no way of itself, requires any attention. The color, however, often reaches 150, and, usually being accompanied by very low alkalinity, causes at times considerable trouble, and taxes the capacity of the coagulant feeds to the limit. Its entire removal, while desirable, is not absolutely essential, for sterile plates on the effluent can be obtained when the color is quite noticeable.

TABLE II.—CHEMICAL ANALYSIS OF RAW AND FILTERED WATER.

	November		December	
	Raw.	Filtered.	Raw.	Filtered
Color	50	0.0	45	0
Turbidity	0	0	5	0
Odor	1e	0	1v	0
Nitrogen as—				
Free ammonia	0.042	0.048	0.064	0.032
Albuminoid ammonia	0.114	0.112	0.082	0.032
Nitrites	0.000	0.001	0.002	0.001
Nitrates	0.400	0.480	0.24	0.480
Chlorine	6.0	6.0	5.5	5.5
Iron	1.1	0.1	0.5	0.0

All figures given are in parts per million.

Bacterial counts are made daily, on agar at 37½° C., of the raw water, the filtered water and of the water after being treated with hypochlorite. Tubes of lactose bile are likewise inoculated with known quantities of the same samples, to indicate the probable presence of intestinal bacteria. The raw water always shows B. coli in 1.0 c. c. and often in 0.1 c. c., but the effluent has never given a positive test in 10.0 c. c. The percentage of reduction is not as large as in most plants, due to the fact that the bacterial content of the raw water is low, often running only 25 or 30 per c. c. With such a condition existing in the raw water, it is impossible to show a high percentage of removal, even though the effluent usually contains less than 5 bacteria per c. c. In considering the quality of the filtered water it is therefore necessary to consider the content of the effluent as well as the percentage of bacteria removed. This is a condition very difficult to explain to the average consumer. He hears that a certain plant gives a purification of 99 per cent, and immediately states that water is "99 per cent" pure. He can not understand how the water from a plant giving 90 per cent efficiency may be better than that from a plant giving much higher efficiency.

Tests for alkalinity (reaction with lacmoid) are also made daily and sometimes oftener in order to properly regulate the quantities of alum and soda, a procedure made necessary by the very low and varying alkalinity of the raw water.

The average physical and bacteriological results of the daily tests for November and December are given in Table I.

The chemical analysis is as given in Table II.

The treated water as it leaves the rate controller falls into the clear water basin located beneath the filters in the manner indicated in the accompanying cross-section of the filters, Fig. 2. This basin has a capacity of 200,000 gals., and extends far enough beyond the end of the filter house to allow four additional filter units to be built on its roof at some future time should the water consumption make such addition necessary. The construction is of gravel-concrete, reinforced with steel rods. The ground water on the outside of this basin stands nearly to the level of the floor of the filters, and although no water-proofing material was used in the construction, the basin shows no signs of leakage, even though the walls were cracked by carelessness during construction. After the basin proper was completed, and before the excavation had been backfilled or the superstructure erected the contractor allowed the ground water to rise with the basin empty, floating it off its foundaion, and opening up slight cracks in the end valls. Fortunately the basin settled back into position and the cracks closed so that they ould not be located.

PUMPING PLANT.

The pumping engines draw from the south nd of the clear water basin and pump direct nto the city mains, the surplus going to a 2,-00,000 gal. (available capacity) reservoir loated at the east end of the city. The umping plant consists of two cross compound umping engines of the crank and flywheel ype. The larger one, a 17x34x14x30-in. Snow ump had been in service at the old pumping tation for several years. It has a capacity of ,000,000 gals. per 24 hours and showed a uty when first installed of 111,000,000 ft.-lbs. er 1,000 lbs. of steam. The other, a new 14x 8x10½x24-in. Platt Iron Works pump has a apacity of 3,000,000 gals. per day at a speed f 62½ r. p. m. Its duty on test was 125,000,-00 ft.-lbs. per 1,000 lbs. of steam, under the ollowing conditions:

verage steam pressure, lbs	123.9
verage receiver pressure, lbs	10.5
verage vacuum, ins	27.28
verage head pumped against, ft	196.8
verage suction, ft	8.8
verage total head, ft	205.1
verage R. P. M	61.8
ctual rate of discharge, gals. per 24 hrs.	2,949,900
uty per 1,000 lbs. of steam, ft.-lbs.	125,174,000

The steam used by the vacuum pump was ot charged against the engine.

Under working conditions 430 gals. of water re pumped per pound of coal. This includes he coal burned to furnish steam for the lower and other small engines at the plant.

As shown in Fig. 1, the suction of the 3,000,-00 gal. pump is arranged to take water direct rom the main supply pipe if necessary in ase of an emergency such as a large fire.

The steam for the pumps is supplied by two 25 HP. water tube boilers manufactured by e McNaull Boiler Works. As with the umps, only one boiler is used at a time, the ther being held in reserve. The boilers were ested at the same time as the new pump, the ontract price being dependent upon the efficiency shown. The details of the test run are riefly as follows:

rate surface, sq. ft	27
eating surface, sq. ft	1,240
Coal consumption:	
er hour, lbs	512.1
er sq. ft. of grate per hour, lbs	18.9
ry coal per hour, lbs	508.0
Evaporation:	
emperature of feed water, deg. F.	46
ean gauge pressure, lbs	128
vaporation per hour, lbs	4,357
quivalent from and at 212°, lbs	5,219
quivalent from and at 212° per lb. of dry coal, lbs.	10.41
Efficiency:	
alorific value 1 lb. dry coal, B.T.U.	11,825
otal heat above 46° F. 1 lb. of steam at 128 lbs. gage, B.T.U.	1,178
bs. dry coal per hour	508.0
bs. dry steam per hour (128 lbs. gage)	4,357
fficiency, pct.	68.2

The fact that the storage reservoir is a mile d a half from the pumping station made it ecessary to arrange a device for indicating the pumping station the height of water in e reservoir. A suitable device was finally und and arranged to fit the conditions. Three ires were run from the plant to the reservoir, e return being taken by the ground. A float the reservoir connected to a cord running er a pully and counterweighted rises and lls with the level in the tank. A button on e cord closes one circuit when the level aches either maximum or minimum, ringing bell in the engine room by means of a relay. he pulley over which the cord runs is connected by means of ratchets so as to close one the other circuits once for every 1 in. of ll of the float and to close the third one r every 4 ins. rise of the float. The closing the circuits excite magnets at the pumping ation lifting their armatures and turning an dicator on a dial by means of a pall and tchet, one circuit turning the indicator in e direction and the other in the opposite. he current is supplied by open circuits and lls.

The cost of the plant was as follows.

Coagulation basin	$ 5,820
Filters	17,960
Clear water basin	13,256

Making the cost of filter plant proper $37,000 or $12,300 per 1,000,000 gals. capacity.

Pumping station	$30,002.53
Coal bunker	3,919.00
Pumps and boiler	21,934.00
Chimney	3,020.00

A total of $58,875 for the pumping station or about $19,600 per 1,000,000 gals. capacity including duplicate machinery.

During the period that the plant has been in operation the cost per 1,000,000 gals. of water supplied has been $31.40. This figure includes interest and sinking fund and maintenance of the distribution mains.

Digest of Court Decisions of Controversies Between Water Companies and Individuals Claiming Damage by Polluted Water.

A study of court decisions adjudicating controversies between water companies and individuals claiming damage by reason of the sale or distribution of polluted water, is interesting chiefly in showing the different viewpoints held by courts of such questions at the present time from those held 20 years ago. Dr. H. E. Barnard, Chemist of the Indiana State Board of Health and Chairman of the Committee on Legislation and Legal Decisions of the Indiana Sanitary and Water Supply Association presented as his annual report before that body a digest of the legal decisions of the character here under consideration. The compilation was carefully made and should be preserved for reference by all water departments. The report follows:

Rights of property from the opinions I have been able to collect appear to have once been paramount over the rights of individuals. While the law may have been as carefully construed than as now, a study of the decisions shows clearly that the courts did not interpret the law in the same light then as at present.

It has been somewhat surprising to me to find how few decisions have been handed down by the higher courts involving the liability of water companies who have distributed polluted water.

The first case to which I shall call attention is the classic Buckingham v. Plymouth Water Company.

Buckingham vs. Plymouth Water Company. 142 Pa. 221.—Where a water company drew water from the river which was infected in its course through a town in which there was typhoid fever, the company being ignorant of the disease, and the river banks had recently been inspected by the company,, it was held that it was not liable in damages for the death of one who drinks the water and contracted typhoid fever.

This case grew out of the historic epidemic at Plymouth,Pa., where out of a population of 7,000 there were about 1,200 cases of typhoid fever. The plaintiff proposed to show that dejecta of typhoid fever patients living along the banks of Toby's Creek was carried into and down the creek to the Susquehanna River and emptying into it a mile and a half above the intake of the Plymouth Water Company. This testimony was held to be irrelevant unless it were shown that the company had knowledge of these typhoid fever cases along the alleged source of its water supply. Since the evidence did not show that the defendant company knew of the case of typhoid fever or of the fact that the dejecta of the patient had been thrown upon the ground near the stream, but does show that the superintendent of the company from time to time went along the bank of the stream, the court deems it its duty to hold that the plaintiff has not shown that the sickness of his children was caused by the negligence of the defendant.

Danaher, Adm'r City of Brooklyn, 119 N. Y. 241.—In an action to recover damages for the death of plaintiff's intestate alleged to have been caused by drinking unwholesome water from a well, used gratuitously by the public, belonging to defendant, and under its control, a city is not bound to make a chemical examination of the water of free public wells maintained by it for the purpose of ascertaining whether or not it is pure and wholesome where it has no notice that it is unwholesome, and it is not liable for injuries caused by drinking unwholesome water from a well maintained by it, in the absence of wilful misconduct or culpable negligence on its part with reference to the water.

The burden upon the city is sufficient if it be held to the responsibility of keeping the pumps and wells in order and clean, and if it be made liable for any injury resulting from the use of impure waters from the wells after it has had notice of their dangerous qualities. and an opportunity to remove the danger. A higher degree of diligence as to whether apparently pure and wholesome, agreeable to the taste and in common used by the public without complaint, would be unreasonable. The court holds opinion that the city is under no burden to make a chemical examination of the waters of the public wells for the purpose of ascertaining whether they were pure and wholesome.

The court further says that this case is not like the cases where a city creates or permits a nuisance, or turns a stream of mud or water upon the premises of private individuals, in which case it is held responsible for the nuisance which it creates or permits.

Green, Adm's v. Ashland Water Company 101 Wis. 258.— A distributor of water under a public franchise will be liable for injury to consumers without fault on their part from using the water furnished, if it knows, or from the situation ought to know, that the water it is distributing is dangerous for domestic use for some cause not discoverable ordinarily by the use of reasonable care, and if it fails to disclose such danger to them, but if the source of supply be contaminated with sewage for a long period of time, causing epidemics of typhoid fever annually in the community for several years, and the fact in that regard be there notorious and a matter of common knowledge, the presumption is that members of such community of ordinary intelligence have notice of that situation; and, in the absence of evidence to the contrary, that presumption will prevail and prelude a recovery by a person injured by the use of such water on the ground of his contributory fault.

This was an action to recover damages for the death of a user of the water alleged to have been caused by negligence on the part of the Ashland Water Company in that it failed to extend its intake pipe into the Chequamegon Bay from time to time as it needed to secure water free from sewage contamination and suitable for domestic use. It was claimed that because of this breach of data the water company took water from Chequamegon Bay that contained typhoid fever germs and distributed it to its customers in the city of Ashland one of whom drew such water from a surface faucett at his residence and contracted typhoid fever, of which he died.

From this opinion we must draw the conclusion that at least one court holds that if a water company furnishes water so bad that it is notoriously impure it is exempt from liability. In other words, it is safer to sell a bad water than a water ordinarily good but which under certain stress may be temporarily bad

Commonwealth v. Potter County Water Company 212 Pa. 463.—Where the evidence against a water company, although contradicted, tends to show that the company's line was connected with a mill pond into which the sewage of the town was discharged, and that during periods of low water, and when fires occurred, water from this pond was pumped into the main service lines of the company, and that such water polluted the other water and frequently rendered it injurious and unwholesome and unfit for drinking or cooking, and even for washing, a judgment of ouster on a verdict for the commonwealth on quo warranto proceedings was sustained.

In rendering the opinion the Judge said:

> Nothing within the limits of reason could justify the failure upon the part of the water company to furnish to the public water which was at least reasonably pure and comparatively wholesome.

Lockwood, Adm'r v. Dover, 73 N. H. 209.—A city was held to be liable for the death of a consumer of the water furnished by it, caused by foreign and decayed matter becoming lodged in the city's pipes by reason of its negligence in not cleaning out the pipes.

In the Case of Stein v. State, 37 Ala. 123.—Indictment does not lie against one for supplying and selling unwholesome and poisonous water, in the absence of any knowledge on his part of its unwholesomeness and poisonous quality.

Case of Georgetown Against the Water, Gas, Electric and Power Co.—This was a case in which the corporation undertook to furnish the city a supply of water for domestic use and fire protection from a source furnished by the city. This water became impure and the city petitioned the court to require the defendant company to install a filtration plant. The court held that in the absence of a contract to that effect so long as the city supplied the water supply it could not impress upon the water company the duty of constructing a filtration plant for the purpose of purifying such water.

Peffer v. Pennsylvania Water Co., 221 Pa. 578.—Plaintiff filed a bill against a water company alleging that the water furnished by it was impure and was the cause of typhoid fever, and the evidence showed that the water was polluted, containing bacteria and bacilli. *Held*, that a decree directing the company to provide a sufficient supply of pure water and to file a statement, within the time specified, showing what it proposed to do in order to comply with acts regulating public water supply, was properly rendered.

> In a proceeding to compel a water company to furnish pure water, a company is not required to furnish chemically pure water. It must furnish water ordinarily and reasonably pure and wholesome.

"Ordinarily pure and wholesome water" means water reasonably clean and free from bacteria or other contamination, rendering it unfit for domestic use and dangerous to individuals.

In this case the analysis showed the presence of colon bacilli in proportionally large quantities in the raw Allegheny River water, the number of which was materially reduced after filtration. However, both bacteria and the colon bacilli in dangerous quantities are found in the water after filtration.

It appeared from the testimony that several features, notably the extensive pollution, the prevalence of typhoid fever above the city, the method of increasing the supply at the expense of impaired filtration and the inability of the company to keep the filter beds in good order, that the water furnished by the company is not reasonably pure and wholesome.

In this case the testimony given by physicians, chemists, engineers and eminent filtration experts covering 1,100 typewritten pages, completed the conclusions reached as to the character of the water.

The court further says that modern investigations and scientific attainments demonstrate that water from polluted sources can by proper filtration be made reasonably safe and pure, and therefore the standard of purity as fixed by the well recognized authorities must be secured.

Keever v. City of Mankato, 113 Minn. 55.—A complaint charged that defendant city negligently allowed the supply in its water works system to become polluted, and large quantities of sewage to escape into its water supply, by reason whereof plaintiff's intestate contracted typhoid fever and died. *Held*, on demurrer, that the municipality was liable for its negligence in its private or corporate capacity, and was not exempt because it was carrying out a government function.

Jaggard, judge, says:

> It is obvious that a sound public policy holds a city to a high degree of faithfulness in providing an adequate supply of pure water. Nor does it appear why its citizens should be deprived of the stimulating effects of the fear of liability on the energy and care of its officials; nor why a city should be exempt from liability while a private corporation, under the same circumstances, should be held responsible for its conduct and made to contribute to the innocent persons it may have damaged.

A water company is not an insurer. As said in Green v. Ashland Water Co., 101 Wis. 258, 43 L. R. A. 117, 77 N. W. 722, a mere distributor of water for compensation is not liabie as a guarantor of its quality.

A city is not liable for injuries caused by drinking unwholesome water from a well maintained by it, in the absence of wilful misconduct or culpable negligence on its part with reference to the water. Inid. Affirming 51 Hun, 564, 4 N. Y. Supp. 312.

Liability of water company for furnishing impure water, see Note 81, American State Reports, p. 483.

Cases involving municipal liability for damages caused by pollution of its water supply, see Note (b) Quality, Contaminated water, 61 L. R. A. 88.

Duty of water supply companies to filter water, see Note 24, L. R. A. (N. S.), 303,304

Purity and Quality of Water, see 40 Cyc. 786.

Duty of a water company as to quality of water supply, see 1 Farnham on Waters, Water Rights, p. 829, and cases cited.

Measures of Economy Available to Small Water Works in Selection and Operation of Devices and Machinery.

More than four-fifths of all the water plants of Indiana belong to the smaller classes in which daily pumpages probably average less than 1,000,000 gallons. As compared with the larger systems, these enterprises have relatively few consumers, limited revenues, and proportionately much higher fixed charges and operating expenditures. Therefore, it is easy to realize that the problems confronting such utilities as relate to economy and efficiency are of paramount concern. The smaller the utility, all the greater becomes the necessity for the closest scrutiny into its plant operations for the detection and stoppage of waste, and for economical conduct of its affairs generally.

Mr. E. L. Loomis, superintendent of the Valparaiso Home Water Co., of Valparaiso, Ind., as chairman of the committee on water works devices and machinery, presented a report before the recent annual convention of the Indiana Sanitary and Water Supply Association in which he discussed some of the measures of economy available to small water works in the selection and operation of devices and machinery with special reference to those used in the pump and boiler rooms. The matter here given is taken from that report.

A visit to almost any of the larger plants of the state will disclose the fact that they are not only equipped with all sorts of devices and facilities for the economical conduct of their enterprises, but will show also that they have experienced and technically trained men to manage and supervise them as well. Many of the smaller concerns likewise exemplify a similar degree of thrift and skill. A detailed study of these systems and of their various kinds of operations for plant economy is well worth while, and will doubtless yield abundantly in returns if the information thus acquired is adopted and applied, so far as practicable, to the smaller plants.

As illustrating an experience in pumping economy Mr. Loomis described the conditions existing at the Valparaiso plant and made a comparison of the costs of operating each of the three different types of pumps.

The pumpage approximates 1,000,000 gals. per day, pumping direct against a total head of 85 ft. The principal pump for continuous operations is a cross-compound, condensing engine, with Meyer gear and attached air pumps. Upon acceptance test when it was purchased two years ago this machine showed a duty of 78,570,000 ft.-lbs. per 1,000 lbs. of steam used, pumping against the usual head at normal load.

Assuming an apparent evaporation of 7 lbs. of water to each pound of coal, and a total head of 100 ft., with a working duty of about 70,000,000 ft.-lbs. per 1,000 lbs. of steam, this machine will pump and deliver 1,000,000 gals. of water each day for one month at a steam consumption of 358,071 lbs., thus requiring 25.57 tons of coal, which at $3.50 per ton in bin, costs us $89.49. With the No. 2 reserve pump, of the non-compound, condensing, crank and flywheel type, assuming a working duty of only 50,000,000 ft.-lbs., which really is a trifle too high, for the same pumping operation, 501,300 lbs. of steam would be consumed, requiring 35.80 tons of coal at a cost of $124.30. With the third unit, a simple duplex pump which, needless to state, is rarely placed in service, 752,129 lbs. of steam would be used, or 53.71 tons of coal costing us $187.98.

Thus it is observed that by using the No. 1 pump, for each month we operate we show a saving of $34.81 over pump No. 2, while over pump No. 3 a saving of $98.49 is effected, or, in round numbers, at the rate of almost $1,200 per year.

Such a saving to any small plant in a comparatively short time would be sufficient to buy and install the very best and most economical type of pump which the market affords.

In the case of Valparaiso it is plainly apparent that from the standpoint of even ordinary business economy we had better consign both of our low duty pumps to the scrappile rather than place them in continuous operation. As auxiliary units, for temporary emergency purposes, however, they of course have their proper place and value.

A good, durable, dependable and economical pump for the particular service required, is the very heart of every water works system, and no plant can reach its highest state of efficiency unless its pumping equipment combines all of these fundamental requirements.

The pumping machine for every plant should be designed so as to give the best possible economy at the normal load, but with ample capacity for the maximum requirements, such as the sudden opening of large connections, or where direct fire pressure is furnished at the hydrant nozzle. In nearly all cases operating economy should be considered in making a selection rather than first cost. But the fixed charges on a high duty machine of the Corliss type, by reason of its higher first cost, the greater housing space required and its many complications, may sometimes more than counterbalance the saving over a simpler type, such as the Meyer gear.

While, as heretofore noted, a great saving may be effected by the use of properly designed pumping machines, the same degree of economy is applicable to the boiler room. The kind of boilers, the character of fuel, and methods of firing are all-important questions in considering boiler room economics. For every plant the boilers should be in duplicate units and, like the pumps, of ample capacity for the peak load, though not too large for the normal uses. As safeguards, the steam lines from the boilers to each pumping unit should be as nearly separate as possible. Obviously also should the boiler feed-water systems be entirely separate and independent.

In determining the kind of coal to be used it has been the experience at Valparaiso that dependence should not be placed upon the basis of heat units alone, but that the coals should be tried out, not merely for a few days under the most favorable firing conditions, but thoroughly tested under the everyday working conditions just as they actually exist. Under such test, the coal evaporating the most water for the dollar manifestly should be the kind to adopt. Moreover, with propriety, and sometimes with profit as well, coal bought on contract may be tested from time to time as a check upon its continued good quality. For this purpose where the amount consumed will not warrant the purchase of a good bomb calorimeter, the lead oxide reduction method may be used, such an outfit costing but a few dollars and the results being sufficiently satisfactory for purposes of comparison.

Where the coal storage room is located adjacent to a railroad siding, if used in quantities sufficient to justify, coal conveying machinery may effect a substantial saving over the usual hand operations.

With good boilers, the right kind of pumping machines and the best kind of coal in stock for the purpose, there yet remains the continuous, all-important, every-day question of operations, of fueling, and of so conserving the fuel energy stored away in the coal as to utilize and transmit the same direct to the engine throttle in quantities as needed, rather than as we too often observe, carried up the smoke-stack and wasted. Fuel conservation is one of the most difficult of all perplexing questions confronting every power plant. Even the most careful firemen need every practicable facility with which we can surround them if the highest results are to be continuously maintained.

As aids in this work, involving as it does one of the most conspicuous items in operating costs, the best patterns of shaker and dumping grates, and also automatic damper regulators, may be used with profit. If the amount of coal consumed will not justify the purchase of a CO_2 recorder, a boiler efficiency meter with Orsat apparatus will be found to furnish a valuable guide.

Steam flow meters may be had at small cost for attachment to the different steam lines, and when installed will furnish desirable information as to the amount of steam conveyed to the different units or heating systems. Another device of value though of much greater cost, is the Venturi meter, which if placed on the pump discharge line as a check on the engine counter, will often reveal unsuspected pump slippages needing correction.

A common and often unconsidered source of loss in many small plants is leaky valves. A valve reseating machine should be in every power house and by its use every valve kept always in proper repair. Suitable equipments for the various repairs common to all plants from time to time are not only necessary but money-saving assets in times of their need.

Oiling arrangements frequently are not given the attention which their importance would seem to deserve. The system employed should be such that the oils may be saved, refiltered and used again. With such saving they may be used more freely than otherwise, thus safeguarding against hot bearings from time to time, with their consequent annoyance, or possible interruption to the service.

In connection with the operation with filtration systems a good serviceable laboratory would seem to be indispensable for the necessary tests. Cities without such systems and having water at times of questionable quality will find apparatus for the use of hypochlorite of lime to be an inexpensive as well as a most valuable acquisition.

By the use of steam and water pressure recording gages, if supplemented with accurate hourly notations of coal consumed and water pumped, the engineer and manager are afforded definite knowledge concerning the workings of the plant, and also furnished a permanent record of the same.

So well known is the value of the water meter that it would seem to be superflous to make mention of it here And yet the fact remains that in Indiana there are a number of plants operating almost, if not entirely, upon flat-rate bases. As a conserver of water this device is second to none, and its value is closely followed by the meter testing machine. For leakage inspections the detectorphone and pitometer are both widely recommended.

For the office there are many devices worthy of adoption and use, such as the adding machine, addressograph, card indexes, filing apparatus, loose-leaf systems, etc. An accurate map showing definitely the location of all mains, valves and service boxes is of inestimable value and should be deemed an indispensable part of every system. A few well-selected technical books and the best water works journals will prove to be not only sources of information and pleasure, but of actual profit as well. For articles of especial interest, should one wish to dispense with indexes, a scrap-book might be used to good purpose

Railways

Suggested Design for Reinforced Concrete Trolley Line Pole.

A sample design of reinforced concrete trolley pole recommended for trial by a committee of the American Electric Railway Association is shown by Fig. 1. The pole has a 2-in. taper on each dimension. The deflection is computed from the formula.

$$x = \frac{38\ Pl^3}{E_s\ b^4}$$

in which,

x = deflection in inches 18 ins. from top of pole, which is point of application of load.

P = pull in pounds applied 18 ins. from top of pole.

l = length in inches between point of application of load, P, and the surface of the ground (6 ft. from the base end of the pole)

b = length in inches of side of base

E_s = coefficient of elasticity of reinforcing steel assumed as 30,000,000.

This formula is based upon assumptions that (1) The 6-ft. embedment in the ground is absolutely fixed; (2) the concrete cracks on the tension side to the steel; (3) the reinforcement constitutes 4 per cent of the cross sectional area; and (4) the ratio of the coefficient of elasticity of steel and concrete is 12. The following are extracts from the specifications:

The area of the reinforcing rods shall constitute practically 4 per cent of the base area of the pole, and shall be made up of four reinforcing bars placed as shown in Fig. 1. The modulus of elasticity of the reinforcing steel shall be at least 30,000,000. The longitudinal reinforcements will require stirrups at intervals of from 20 to 50 diameters of the reinforcing rods.

These stirrups may be ¼-in. rods or 1/16 in. by ½-in. straps, and should be looped around the four rods with a few inches extension of the ends to insure anchorage. By using sufficiently heavy stirrups and giving a complete turn about each longitudinal rod, the reinforcement can be formed into a self supporting frame that will require only an occasional wooden spacing block to hold it in position while concreting, the blocks to be removed as the concreting progresses.

The concrete shall be made of Portland cement meeting the test requirements of the American Society of Mechanical Engineers, of clean, sharp sand and crushed stone, which shall be clean and free from dust and not more than ¾ in. in its largest dimension. Clean washed gravel of the same size may be used in place of stone. The mix shall be of a richness of at least 1 part cement, 2 parts sand, and 4 parts stone.

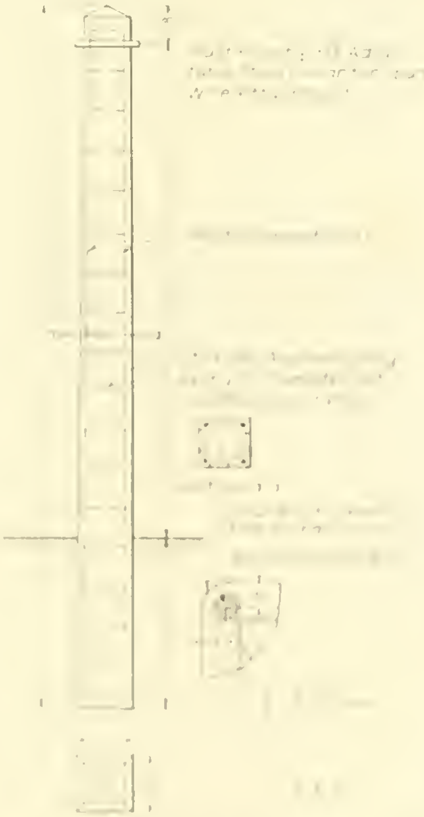


Fig. 1. Design for Reinforced Concrete Trolley Pole.

Standard Railway Property Maps and Profiles Prescribed by the Interstate Commerce Commission.

Standard railway property maps and profiles are required before Feb. 1, 1915, from all railways by the Interstate Commerce Commission. These maps are to show railway property as of June 30, 1914. Also it is required by the Commission that similar standard maps shall, within six months after their completion, be provided of all extensions, improvements or other changes. Typical standard maps are given by the accompanying drawings and the following specifications for such maps as issued by the Commission are reprinted:

1. GENERAL.

1. Intent.—In order that the Interstate Commerce Commission may investigate, ascertain, report, and record the value of property of railway carriers as it now exists and as it hereafter may be extended, improved, or changed, it is essential that certain maps and profiles shall be prepared by the carriers and filed with the Commission.

It is not the intent of the Commission to require the unnecessary construction of maps and profiles. All maps and profiles, both old and new, must be furnished upon sheets of the standard sizes and upon material of the kind specified and they must be produced or reproduced by the process specified.

All new maps and profiles, whether covering new construction or old construction, must be strictly in accordance with these specifications.

Where maps and profiles already in existence contain in the opinion of the Commission the necessary information in such form that it is reasonably available, these will be accepted.

For the purpose of ascertaining to what extent their present maps are acceptable carriers may bring together at their principal engineering offices such maps and profiles as they desire to tender. Thereupon the Commission will at once examine the same and will indicate in writing what are acceptable in their present form and what changes or additions should be made in order to make others acceptable.

Except in case of existing maps and profiles

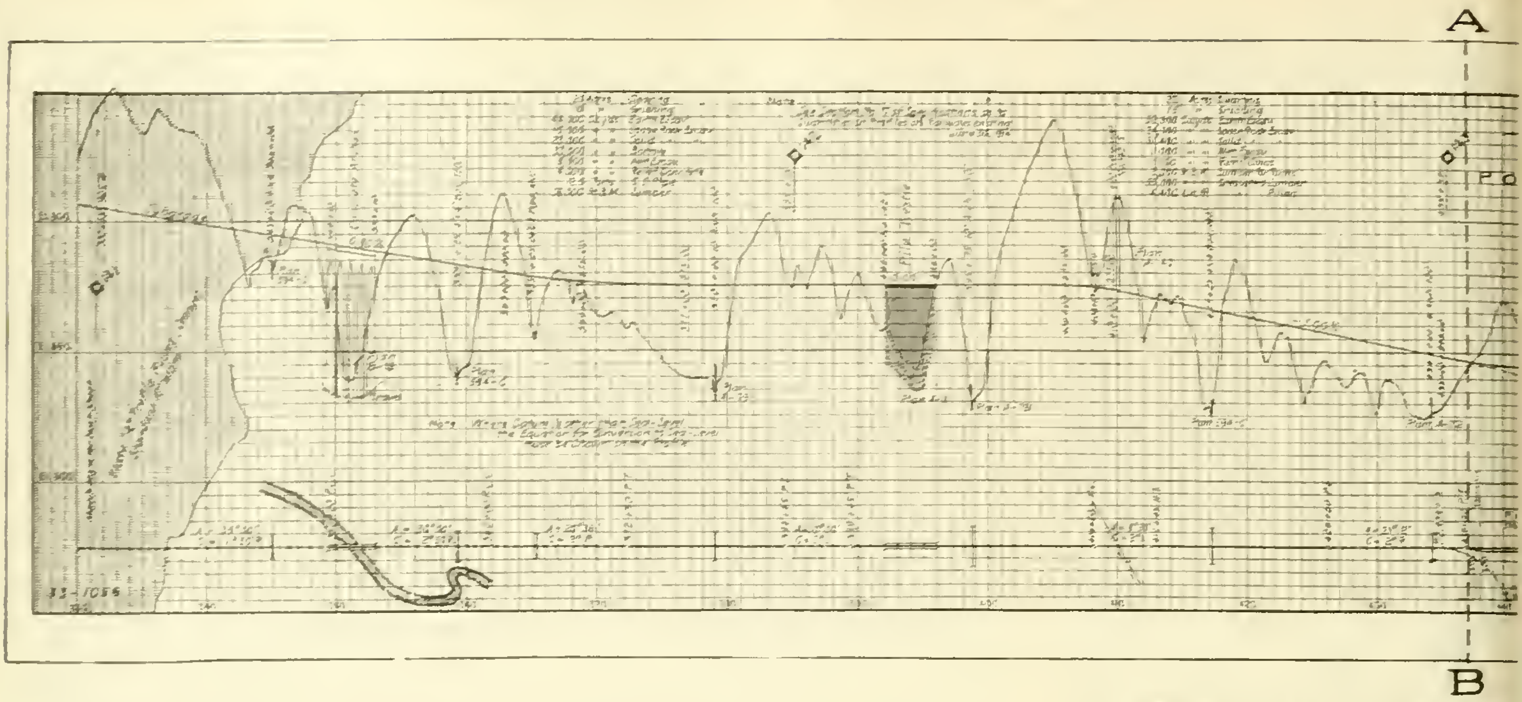


Fig. 1. Standard Railway Property Map

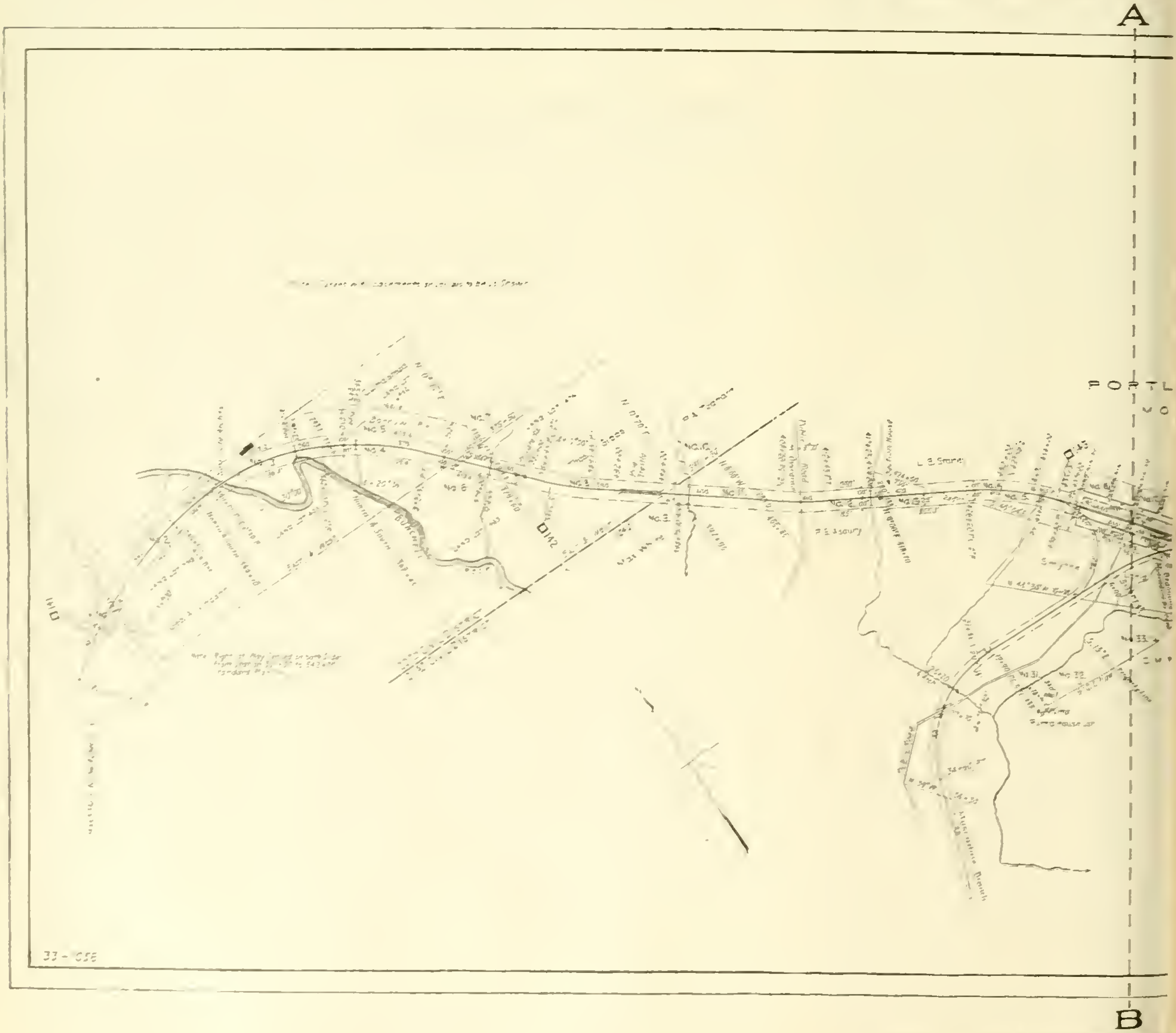
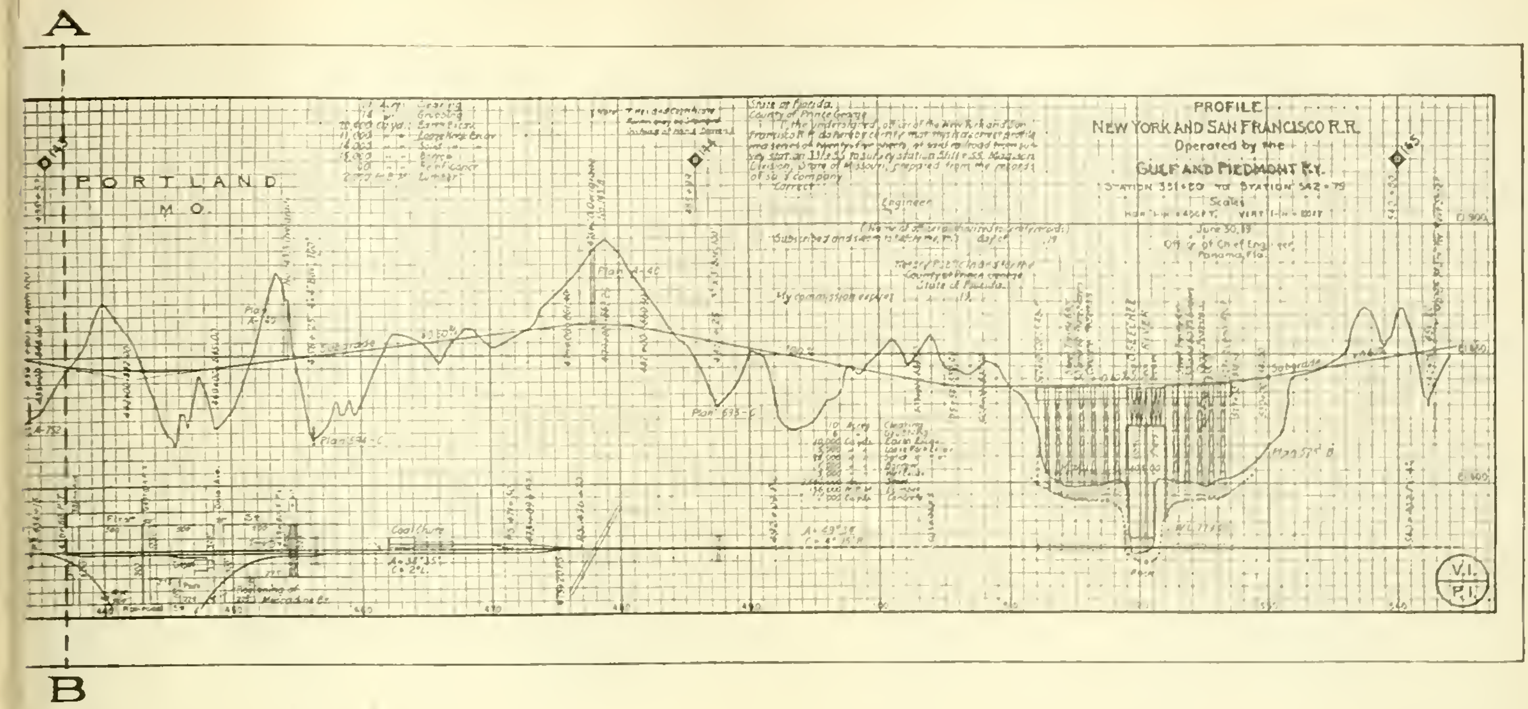
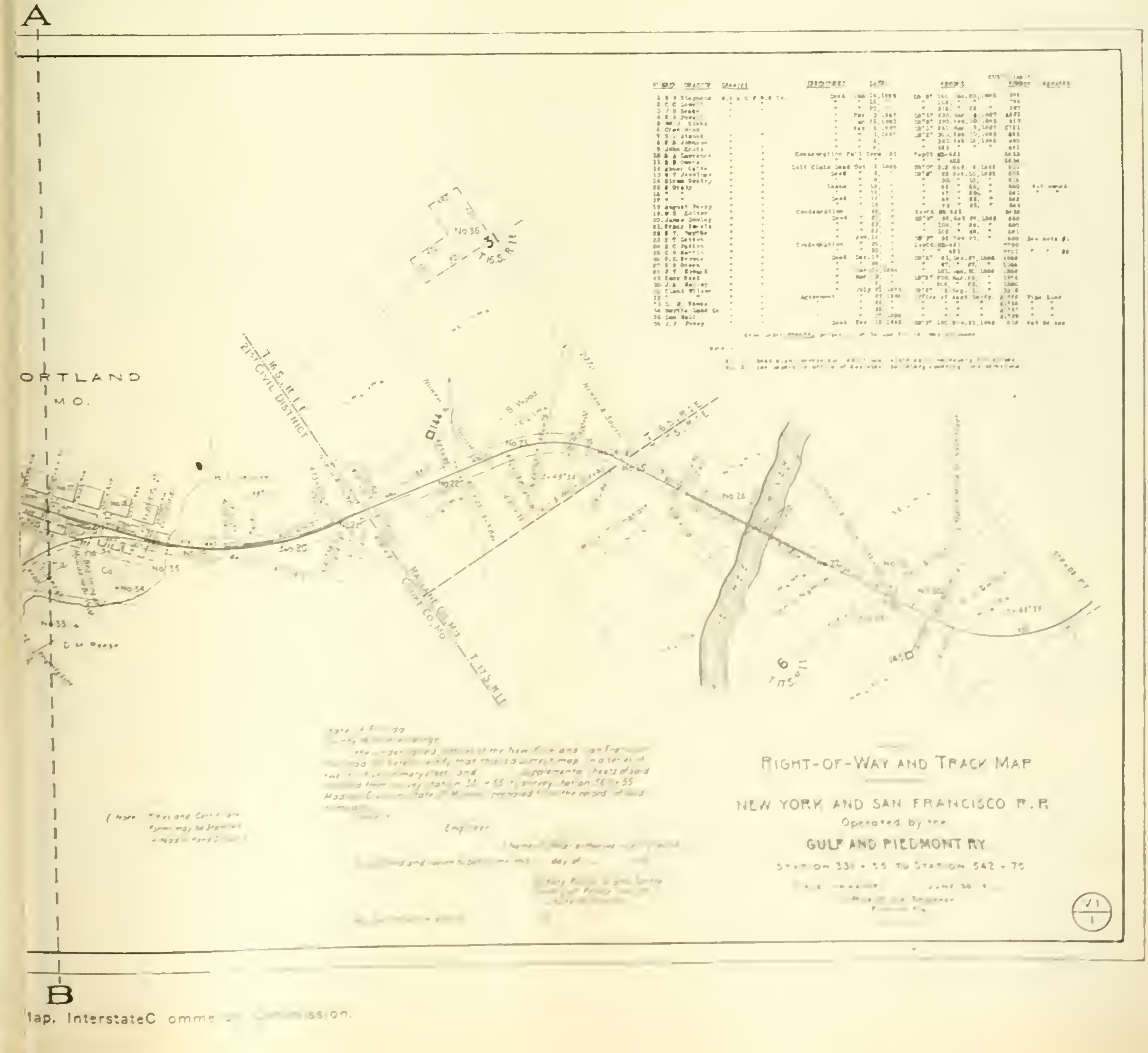


Fig. 2. Standard Right of Way and Track



Profile, Interstate Commerce Commission.



Map, Interstate Commerce Commission.

accepted or modified as above, these specifications must be strictly followed.

II. MAPS AND PROFILES REQUIRED.

2. Classes and Titles.—Three general classes of drawings shall be made by the carriers and filed with the Commission, viz.: (1) Right-of-Way and Track Map. (2) Station Maps. (a) Maps showing all lands, separately from improvements, when this is necessary for clearness. (b) Maps showing tracks and structures and external land boundaries. (3) Profile.

3. Description and Purpose. The Right-of-Way and Track Map shall be a true horizontal projection of the right of way, tracks, and other structures, platted continuously between district or terminal points.

The Station Maps shall be a supplement to the above for terminals and other locations where the property of carriers is so extensive and complicated that it cannot be clearly shown on the Right-of-Way and Track Map. The Station Maps shall be made in two separate sets, one showing details as to lands and the other the tracks, structures and external land boundaries, except that where practicable to show clearly on one map all information specified hereinafter this may be done.

The Profile shall be a vertical, sectional view on center line of track (or other railway base line) on an exaggerated vertical scale, and shall show the features of the railway track substructure which can best be indicated in vertical projection; also such other detail information as is hereinafter more fully set forth.

III. MATERIALS.

4. Cloth.—All maps and profiles shall be made by hand or by a lithographic process, approved by the Commission, on the best grade of tracing cloth (Imperial or its equal).

5. Ink.—The ink used for making maps and profiles shall be the best grade, black, waterproof, and permanent india or printer's ink. The profile ruling shall be printed in orange (colored) ink where hand tracings are furnished.

IV. FORM OF MAPS AND PROFILES.

6. Size of Sheets.—The Right-of-Way and Track Map shall be made in sheets 24×56 ins. A plain, single-line border shall be drawn on each sheet, dimensions inside of which shall be 23×55 ins.

The Station Maps shall be made in sheets 24×56 ins., with border line as above. When more than one sheet is required to show a station property, the plat shall be made upon "matched marked" sheets in such manner as to require a minimum number.

The profile shall be made in sheets 12×56 ins., with border. The size inside of border lines shall be 10×55 ins.

7. Scales.—The Right-of-Way and Track Map shall be made on a scale of 1 in. equals 400 ft., or 1 in. equals 200 ft.

The Station Maps shall be made on a scale of 1 in. equals 100 ft., or in complicated situations 1 in. equals 50 ft.

The Profile shall be made on standard plate A, and on scales of: Vertical, 1 in. equals 20 ft.; horizontal, 1 in. equals 400 ft.

8. Symbols.—The symbols used on all maps and profiles shall be the standards recommended by the American Railway Engineering Association, insofar as they may be applicable.

9. Lettering.—All lettering on maps and profiles shall be in plain, simple style.

10. Arrangement of Data.—The Right-of-Way and Track Map sheets shall be made with the zero or lowest numbered station at the left side of each sheet and shall be platted continuously from left to right. Where the use of this method would involve the abandonment of established survey station numbers of a railway, the platting may be done in such a way as to preserve them, provided the maps or profiles for any given main line or branch are continuous in same direction between termini of main line or branch. The general direction of the center line of track shall be as nearly as possible parallel to and half way between the long sides of sheets, so that the maximum space each side of platted right-of-way lines may be available for showing adjacent topography and property lines

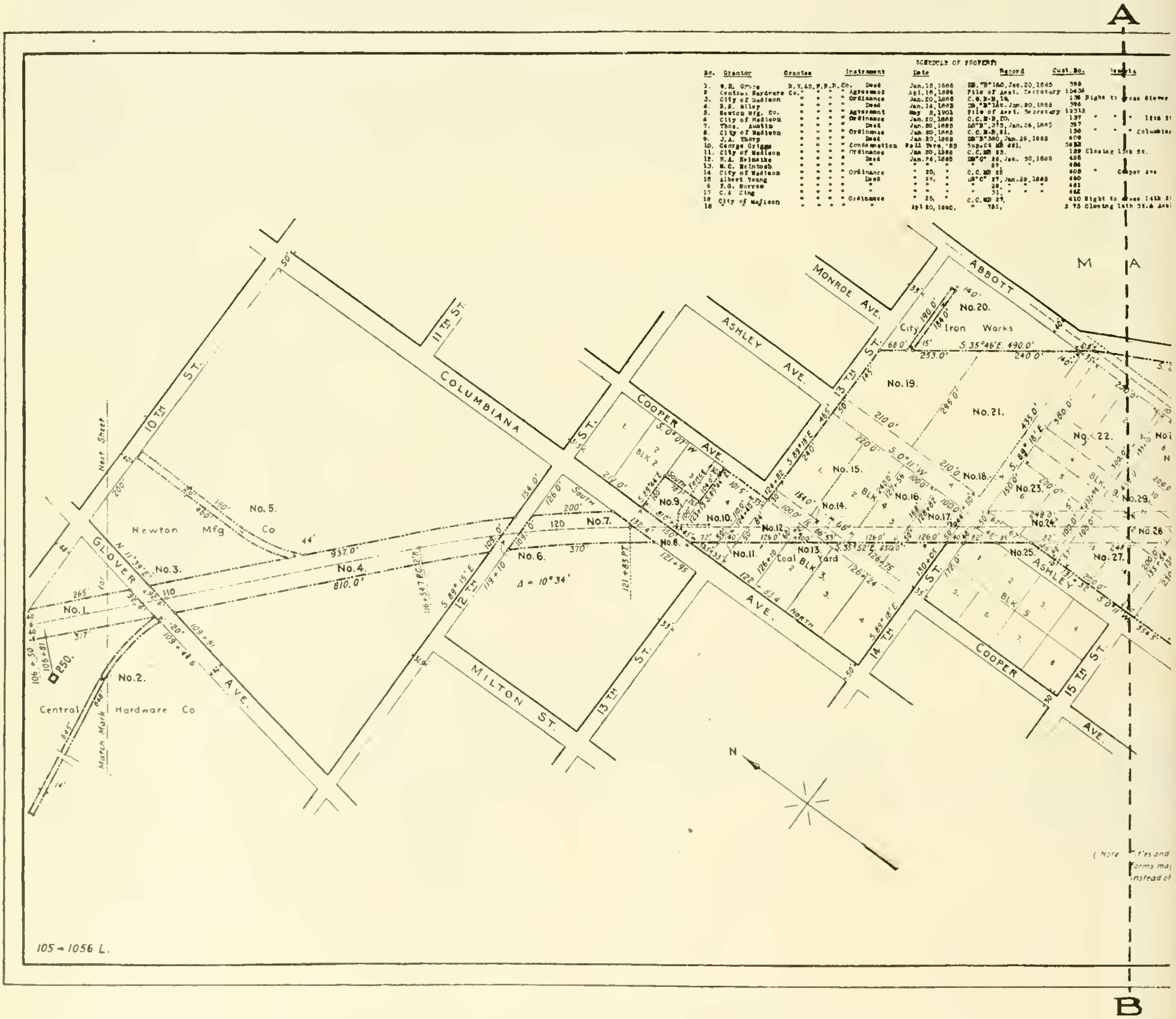
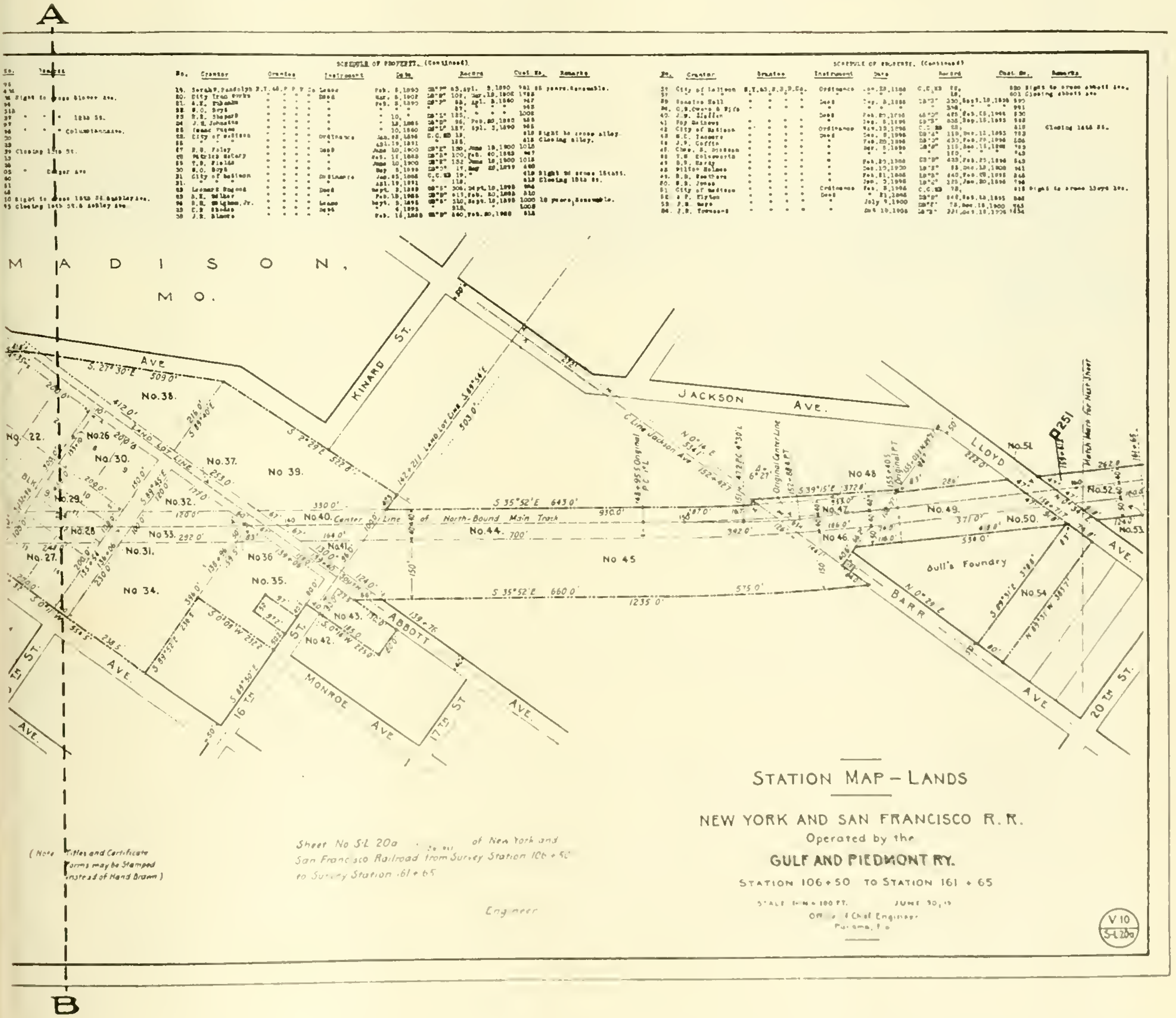


Fig. 3. Standard Station Lands Map.

and for making notes as to physical property. The maximum length of main roadway represented on any one sheet (between "match marks") shall be 4 miles, if scale is 1 in. equals 400 ft., or 2 miles if scale is 1 in. equals 200 ft.

The Station Maps shall be made as prescribed above for Right-of-Way and Track Maps.

The Profile shall be made so that any serially numbered sheet shall cover the same portion of the railway as the like serially numbered sheet or sheets of the Right-of-Way and Track Map. Platting shall be done as specified above. The 2½-in. space immediately above the lower border line shall be used for track alignment and topographic data. The remaining 7½-in. space shall be used for platting the profile in such a way as to most economically utilize the space.


A
B
MADISON, MO.
SCHEDULE OF PROPERTY (Continued)
AVE
KINARD ST.
JACKSON AVE.
LLOYD
ABBOTT
MONROE AVE
17 TH ST
16 TH
BARR AVE
20 TH ST.
Center Line of North-Bound Main Track
Bull's Foundry
No. 38.
No. 37.
No 39.
No. 40.
No. 44.
No 45
No 48
No. 49.
No. 50.
No. 51
No. 54
STATION MAP – LANDS
NEW YORK AND SAN FRANCISCO R.R.
Operated by the
GULF AND PIEDMONT RY.
STATION 106+50 TO STATION 161 + 65
V10
S-L20a
(Note Titles and Certificate Forms may be Stamped instead of Hand Drawn)
Sheet No S-L 20a of New York and San Francisco Railroad from Survey Station 106+50 to Survey Station 161+65
Engineer

Interstate Commerce Commission.

11. Cardinal Points.—On all Right-of Way and Track Map sheets and Station Maps an arrow showing the true north and south line (as nearly as can be ascertained from existing records) shall be placed. This arrow shall be not less than 3 inches in length and shall have the letter "N" marked at its north end.

12. Indexing.—For each series of Right-of-Way and Track Maps there shall be made a small skeleton index map on a scale of not less than one-fourth inch equals 1 mile. Where practicable this index may be placed on any vacant space of the first sheet of a series, and where made on a separate sheet it shall be 24×56 ins. This index map shall show by outline with file numbers therein the sheets of a series, the name of main line division or branch line, the principal cities or towns, and the beginning and ending station numbers of series, and any other information carrier may elect to place thereon.

All Right-of-Way and Track Map sheets and Profile sheets shall be numbered serially, beginning with sheet 1. The sheets representing valuation sections shall form separate series, and valuation sections shall be numbered serially with the letter "V" preceding the number. The letter "P" shall precede the serial number or numbers on the profile sheets. Index numbers shall be in lower right-hand corner of the sheet and inclosed in plain, single-line circle 1 in. in diameter. Valuation numbers shall be in the upper half of circle and sheet number below with a straight line between.

The Station Maps shall be given the same serial number preceded by the letter "S" as the sheet of the Right-of-Way or Track Map which they supplement.

In case a Right-of-Way and Track Map sheet is supplemented by more than one Station Map, a subscript letter should be used after the number, e. g., S 32a, S 32b, etc., where land and track features are combined; S-L 32a, etc., where land only is shown; and S-T 32a, etc., where track features only are shown.

On the Right-of-Way and Track Map sheets references to all Station Maps shall be shown by outlining limits of Station Maps and giving the number of the Station Map sheets.

The carrier's file number shall also be placed on all map and profile sheets in the lower left-hand corner.

13. Title.—The title shall be placed as near the lower right-hand corner as practicable. The following information shall be given therein:

(1) Class.
 Right-of-Way and Track Map.
 Station Map.
 Profile.
(2) Corporate name of the railway.
(3) Name of operating company.
(4) Name of railway division or branch line.
(5) Beginning and ending survey station numbers on sheet.
(6) Scale or scales.
(7) Date as of which maps or profiles represent the facts shown thereon.
(8) Office from which issued.

14. Certification.—A certificate as to the correctness of all maps and profiles shall be printed and executed on the first sheet of each series, and each of the other sheets of the same series shall be identified as a part thereof. The certificate on the first sheet of each series shall

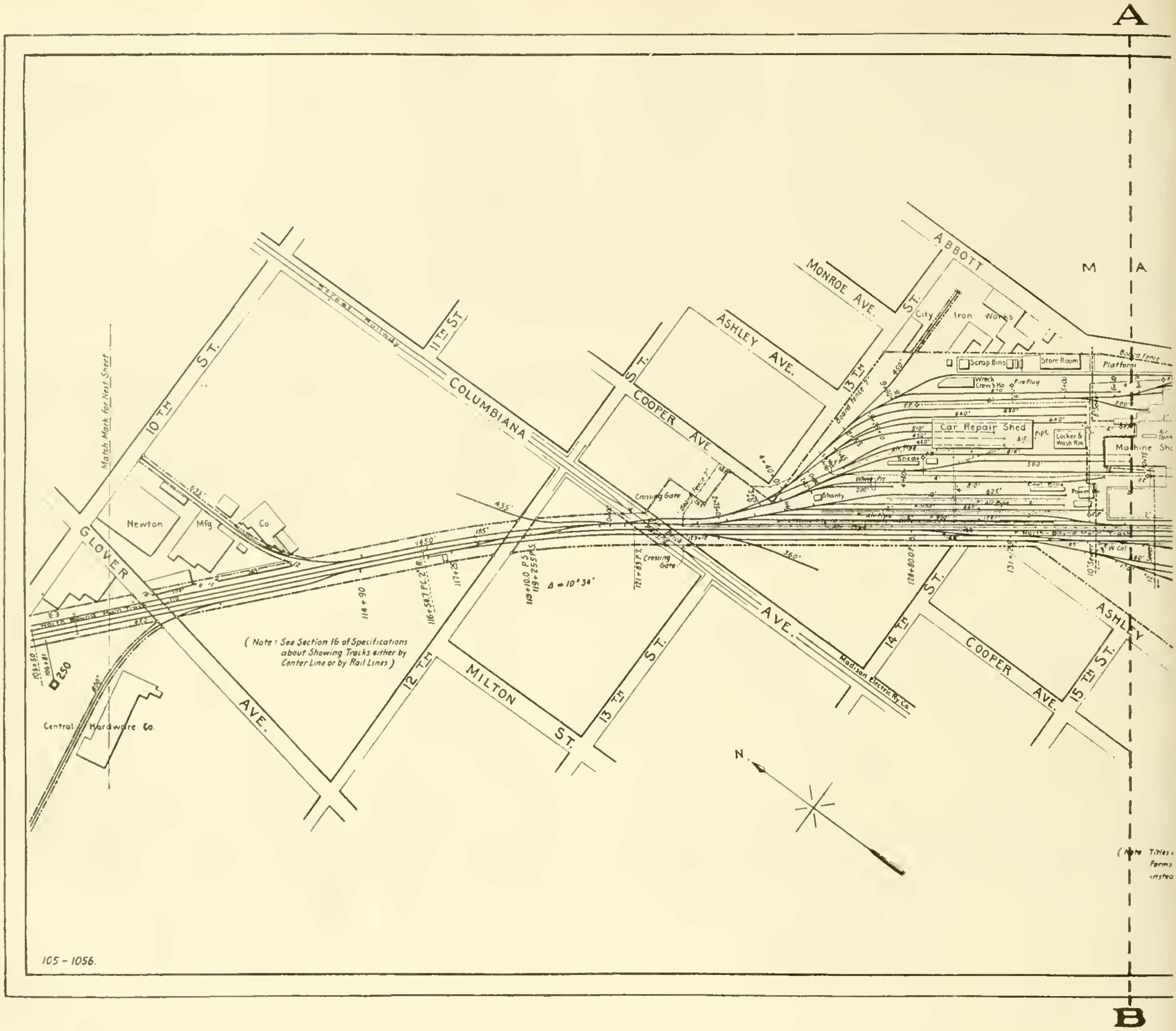


Fig. 4. Standard Station, Tracks and Structures

be placed as near the title as practicable and shall be of the following form:

State of................................

County of ....

I, the undersigned, officer of the ....
(Name of railway company.)

do hereby certify that this is a correct..
(Map or profile.)

in a series of .... sheets, of said railway from survey station ....

.... to survey station ....
(Main line, division, or branch.)

.... State of..
prepared from the records of said company.

....
Engineer.

Correct:

....
(Name of officer authorized to certify records.)

Subscribed and sworn to before me this................ day of ....

....

Notary public in and for the
County of ....
State of ....

My commission expires..

The identification on the other sheets of a series shall be of the following form and placed as near the title as practicable:

Sheet No. .... of .... of ....
(Series.) (Railway, main line or branch.)

from survey station .... to survey station ....

....
Engineer.

V. DATA REQUIRED.

15. On the Right-of-Way and Track Map. On the Right-of-Way and Track Map shall be shown the following data:

(a) Boundary lines of all right of way.—The term "right of way" as herein used includes all lands owned or used for purpose of a common carrier, no matter how acquired.

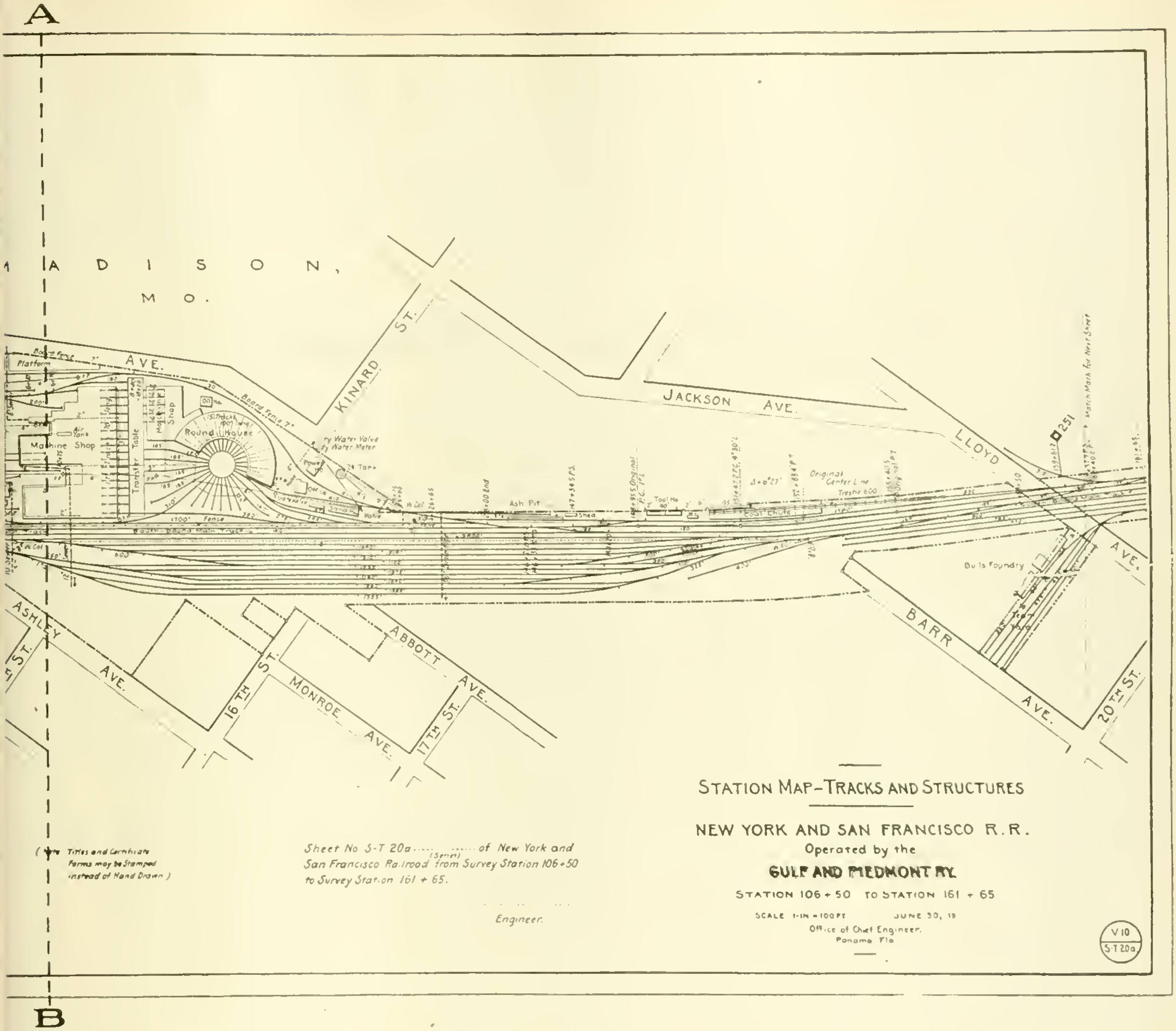
Show: Width of right of way, in figures, at each end of the sheet and at points where a change of width occurs, with station and plus of such points; boundary lines and dimensions of each separate tract acquired; a schedule of deed, custodian's number, the name of grantor and grantee, kind of instrument, date and book and page where recorded. Each tract of land shall be given a serial number and listed serially in the schedule. The schedule shall also include reference to leases to the company, franchises, ordinances, grants, and all other methods of acquisition.

(b) Boundary lines of detached lands.—Where same can be shown clearly. The term "detached lands" as herein used includes:

(1) Lands owned or used for purposes of a common carrier, but not adjoining or connecting with other lands of the carrier.

(2) Lands owned and not used for purposes of a common carrier, either adjoining or disconnected from other property owned by the carrier.

Show: Boundary lines and dimensions; distance and bearing from some point on the boundary line to some established point or permanent land corner, where practicable, and



Map, Interstate Commerce Commission.

separately on the schedule above, the lands not used for purposes of a common carrier.

(c) Intersecting property lines of adjacent landowners.—Where the information is in the possession of the carrier show: The property lines of adjacent landowners, the station and plus of important intersection of property lines with center line of railway or other railway base line, and the names of owners of the land adjacent to the right of way

(d) Intersecting divisional land lines.—Show: Section, township, county, state, city, town, village, or other governmental lines, with names or designations; the width and names of streets and highways which intersect the right of way, and the station and plus at all such points of crossing or intersections with center line of railway or other railway base line.

(e) Division and subdivision of lands beyond the limits of the right of way.—Where the information is in the possession of carrier show: The section and quarter-section lines for a maximum distance of 1 mile on each side of the center or base line of railway where the land has been subdivided into townships and sections; such data as to additions, tracts, streets, alleys, blocks and lots where the land has been divided in some other way than by sections; the distance, where known, from railway-base line to permanent land corners or monuments; and the base line from which the railway's lands were located (center line of first, second, third, or fourth main track or other base line).

(f) Alignment and tracks.—Show: The center line of each main and side track when such tracks are outside the limits covered by the Station Maps and center line of each main track, also inside Station-Map limits; the length, in figures, of all sidetracks from point of switch to point of switch, or point of switch to end of track; all other railways crossed or connecting, and state if crossing is over or under grade, and give name of owner of such tracks, survey station number at even 1,000 scale-feet intervals, and station and plus at points of all main line switches at points of curves and tangents and at beginning and ending points on each sheet, the degree and central angle of curves, and joint tracks and ownership thereof.

(g) Improvements.—Show: Station and office buildings, shops, engine houses, fuel stations, water stations, etc. (owned by the carrier), in general outline, where it can be done clearly. Also indicate conventionally: Bridges, trestles, culverts, tunnels, retaining walls, cattle guards, mileposts, signal bridges and ground masts, fences by note only, and other principal railway structures owned by the carrier, with general data as to dimensions, and, where practicable, pipe lines, sewers, underground conduits, paving, curbing, or similar works located on the right of way of the carrier or adjoining and owned by the carrier in whole or in part. Give station and plus to all important structures which are outlined above.

(h) Topographical features.—Show: Rivers, creeks, watercourses, highway crossings, etc. Gives names, where known, and when highway crossings are over or under grade, so state.

16. On Station Maps.—The purpose of the large scale Station Maps is to permit the showing of improvements in more detail than is practicable on the right-of-way and track map.

Where the station property to be mapped is extensive and complicated, it shall be delineated on two separate maps.

(1) Shall show all data relating to ownership of lands.

(2) Shall show all tracks and structures and external land boundaries.

Where practicable, without sacrificing the clearness of the map, the two may be combined into one map.

Show all information set forth under items (a) to (h), inclusive, of section No. 15, when inside of Station-Map limits, and all other surface and subsurface improvements owned by the carrier and not hereinbefore noted, as far as may be practicable.

Tracks shall be represented on Station Maps either by center lines or by rail lines.

17. On Profile.—The following data shall be placed on all Profiles:

(a) Roadway.—Show: The vertical projection of the original ground surface on center line of railway; present grade line (top of the roadbed subgrade); rates of grades; elevations (sea-level datum) at all points of change of grade, at each end of sheets and where profile is "broken," at 50-foot (scale) intervals; and the station and plus to points of change of grade and station numbers at each 1,000-foot (scale) interval near lower border of sheet.

(b) Structures.—Show: Bridges, trestles, culverts, retaining walls, tunnels, and other roadbed structures in vertical projection, stating the kind and general dimensions by figures; average depth of penetration of piling in each bent of trestles, or under other structures, by vertical projection; character of, and depth to foundation bed of masonry structures by vertical projection; reference to railway file numbers of the detail standard or special plans by which the structures were built; existing mile posts; and the station and plus of each of the above indicated improvements.

(c) Quantities.—Profiles of railways built after the date of issue of these specifications shall show for each mile a summary of construction quantities to subgrade, including roadway, bridges, and culverts. Profiles of railways built before the issue of these specifications may show, at the option of the carrier, similar quantities in the same summary form.

The summary of quantities shall be in detail, according to the standard classifications of units used by each carrier.

(d) Alignment and track.—Show: On the lower 2½x55-in. space of the profile sheet, the center line of each main track, developed into straight line or lines, with alignment notes of curves stated in figures; the station and plus at points of curves and tangents; and other data, such as passing tracks, depot buildings, water and fuel stations, highway crossings, railway crossings, and important watercourses that will assist in interpreting the profile. For platting transversely a scale of 1-in. equals 200 ft. shall be used.

Roads and Streets

General Methods of Concrete Road Construction in Ohio; Conclusions from Service Records.

The operation of the State Highway Department of Ohio is confined to the construction, improvement and maintenance of certain roads, designated as inter-county highways, which form a system connecting the principal centers of commerce within the State. Being thus the main arteries of travel, they demand an improvement which will bear the burden of traffic of medium and heavy density. No detailed traffic census has been undertaken by the Department and the decision as to the kind of pavement to be used is not wholly dependent on the character and amount of traffic, but in a great measure on the character of materials easiest at hand. Where traffic conditions are the deciding factors, local authorities are consulted as to present traffic, and an estimate made as to the probable growth due to the improvement and the increasing commercial activities. One of the materials employed is concrete and in a paper before the National Conference on Concrete Road Building Mr. H. D. Brunning, Division Engineer, Ohio State Highway Department, describes as follows the methods and results of concrete road construction:

Owing to the wide availability of materials suitable for first-class concrete construction in the state, the concrete road has found its place among the several types of roads used by the Department. The total mileage of concrete roads completed to date is 39.63 miles, and the mileage under construction and let is 25.06 miles, making a total of 64.69 miles. The first concrete road constructed by the Department was completed in July, 1911, and the total mileage of concrete roads constructed, under construction and let, represents about 18 per cent of the mileage of all classes of roads put under contract since that time. It is thus apparent that the concrete road has a great prominence in the road-building art in Ohio, and the demand for such roads is growing day by day. Such demands often come from farmers and other users of horse-drawn vehicles, which seems to eliminate the oft quoted objection that this type of road, when untreated with a bituminous mat, is hard on country-bred horses.

The materials for aggregates of concrete are distributed over the entire state, and nearly every mile of road in the state is within reach of these materials at a reasonable haulage distance. The western half of the state is an area where limestone is abundant, and the eastern part is supplied to a great extent with furnace slag dumps. Again, the northern and western section of the state is covered with a mantle of clay, sand and gravel, resting directly on the hard bedrock. This mantle was brought in and deposited by the great sheet of ice which at one time covered much of North America. The melting of this sheet of ice formed waters, which assorted the glacial materials, and deposited it in beds of clay, sand and gravel. Thus vast beds of gravel remain in the valleys of the Ohio, Muskingum, Hocking, Miami and Scioto Rivers. Some 150 quarries, operating in the limestone belt, and 30 sand and gravel washing and screening plants are shipping their products into the markets of the state. In addition to these, local sand and gravel deposits and limestone quarries are found in sections where railroad facilities are not at hand. Thus it may be said that no section of the state can be found where concrete materials of prescribed character are not available within reasonable hauling distance.

The term concrete roads, as used heretofore, and used hereafter in this paper, means cement-concrete roads. Three types of concrete pavements have been constructed by the Department, namely: Single course plain, two-course plain, and single course reinforced. The standard type is the single course plain concrete pavement laid on the natural foundation, except in the case of road No. 1, where on about one-half of the road a 4-in. slab was laid on a waterbound slag foundation, which is giving entire satisfaction after 18 months' use.

A two-course pavement (Road No. 9) 12 ft. wide was constructed in Lake County during the season of 1913. The aggregate in the base course is unscreened second quality gravel, and those in the wearing course are crushed boulders and bank sand, all found locally. The cost of this improvement would have been excessive had the material been shipped in, on account of the long haul of seven miles from the nearest railroad delivery point. The wearing course was laid before the base course had begun to set, never more than 15 minutes elapsing between the placing of the two courses. A very satisfactory bond was obtained and to date no separation between the course or cracks are visible in the concrete.

Two single course reinforced pavements (Roads Nos. 5 and 20) were constructed in Harrison and Pike Counties in 1911, being the first concrete roads constructed by the Department. In both cases the work is located on roads that are subject to overflow and it was partly on this account that concrete was selected. The concrete made in the weak proportion of 1 part cement, 2½ parts sand, and 5 parts gravel was reinforced with No. 11 wire, woven to a 6-in. mesh. The reinforcing was placed midway between the upper and lower surfaces. Transverse joints were placed every 50 ft. at an angle of 45° with the line of the road. The surface of the concrete was rough floated with a plank templet giving a safe footing for horses. Careful inspection of both roads after about 2½ years' service shows that no cracks are apparent in the road in Pike County, and only four sections of the 1.25 miles in Harrison County were cracked transversely, but the size of the cracks is inappreciable. The joints on both roads show considerable wear, due largely to the oblique direction. In general the concrete, although lean in the proportion of cement, shows very little wear.

The importance of a well-prepared sub-grade in concrete road construction has been recognized in all the work done. The specifications have been closely followed in the matter of uniformly compacting the roadbed with a self-propelled roller, weighing not less than 6 tons; all depressions which develop during the rolling are filled with acceptable material, and re-rolled until the entire roadbed is uniform in cross-section and thoroughly compacted. No special provision for drainage of the sub-grade has been found necessary, as in all cases the sub-soil has not been retentive, and ample drainage is secured through the side ditches. To date no set standard cross-section for the pavement has been adopted. Three types of cross-section have been used: (1) A uniform thickness of concrete over the entire width of the pavement, forming the same amount of crown in the surface of the sub-grade as in the concrete surface. (2) A greater thickness of concrete at the center than at the sides, leaving some crown in the sub-grade. (3) A greater thickness of concrete at the center than at the sides, built upon a flat sub-grade.

No greatly differing results in the types are found in the permanency of the pavements constructed, and it would be difficult to select the type which would exceed the other two in qualifications to withstand the effects of heavy loads and extreme temperatures.

The standard crown is ¼ in. to the foot. This has been found sufficient to insure perfect drainage of a concrete road with the usual small imperfections in the surface, and after the road becomes coated with a mat of dirt. The ability to drain itself with so small a crown at once becomes one of the merits of the concrete pavement.

Defects in the roads constructed do not point to any faults in the details of the design of the cross-section as they do to the details of inspection. To assist in the matter of inspection, a well equipped laboratory is maintained by the Department, and all materials entering into the construction are given thorough standard tests. A very important requirement in first-class concrete construction is to get a sand of good quality, and where sand of a doubtful character is submitted, it is tested for tensile strength, the requirements of the specifications being that the tensile strength of briquettes made with the proposed sand shall not be less than the tensile strength of briquettes with standard Ottawa sand and the same cement. Cement is tested in carload lots.

The proportions now used in all concrete road construction by the Department are 1 part cement, 1½ parts of fine aggregate, and 3 parts of coarse aggregate. The proportions of the mortar are maintained uniformly 1 part cement to 1½ parts of fine aggregate, and the amount of coarse aggregate is varied somewhat for the different classes of material, so that there is an excess of about 10 per cent of mortar over the voids in the coarse aggregate as an advantage in finishing. Although the leaner mix used on the first two roads, namely: 1 part cement, 2½ parts of fine aggregate and 5 parts coarse aggregate, has given fairly good results, it was found necessary to employ the richer mix to assure absolute satisfactory results. A lean mix requires much more care in mixing and placing, and it is in these items where causes of defects and failures in concrete roads are prominent. At least this has been the experience of the Department.

The mixing is done by batch mixers. No "continuous mixers" are allowed to be used.

Before any work of mixing and placing of concrete is started, the engineer in charge determines the proper proportion of the coarse aggregate to secure maximum density and the excess mortar of 10 per cent above the amount required to fill the voids in the coarse aggregate. The materials are measured by methods which will insure uniform proportions at all times. Usually this is done in wheelbarrows, which are gaged accurately as to capacity; but in a few instances the measuring has been done in cubical boxes containing 1 cu. ft. Our greatest trouble in the past has been to maintain a uniform consistency of the concrete, but this trouble has been greatly alleviated since the contractors' forces are becoming more experienced in this class of work. The concrete is made sufficiently wet to require no tamping, but there is a strong tendency among the contractors to make it sloppy, which necessarily results in more or less separation of the coarse aggregate from the mortar. This tendency has resulted in the formation of waves on some of the heavy grades due to the creeping of the concrete. On the heaviest grade it is advisable to use a somewhat drier mixture and this rule has been carried out in our construction wherever possible.

After the surface of the concrete has been shaped by the use of a templet, it is floated with a wooden float in such a manner as thoroughly to compact and seal the surface, and produce a surface conforming to the exact crown. No depression is allowed in the finished work which is greater than ½ in. from a 10-ft. straight edge applied to the surface of the concrete and parrallel to the center line of the pavement.

The specifications provide that within one hour after the surface of the concrete has been floated, a canvas covering shall be placed over it and left in place for at least twelve hours. This provision has not been carried out in all cases, but instead the surface has been kept wet by sprinkling. The sprinkling is continued for at least five days and in most cases a covering of sand, dirt or straw has been used to assist in retaining the moisture. This precaution is absolutely essential to secure the very best wearing qualities of the concrete.

A common point of failure in pavements thus far constructed is in the expansion-contraction joints, which on most of the roads have been placed at intervals of 30 ft., but in some cases, for reasons unknown to the speaker, a greater spacing was made. However, this failure is usually in cases where poured joints were used, and the speaker's observation is that this source of failure has been largely eliminated by the use of tarred felt joints. In one road (Road No. 11) Baker plates have been used and the results are fairly satisfactory, although some wear is apparent, due probably to the imperfect placing of the plates. Transverse cracks do not appear, except in rare cases, where joints are not more than 30 ft. apart, but in all cases these cracks occur in large number where the joints have a spacing greater than 30 ft. On Road No. 14, Sec. 3 of the Piqua-Troy Road, one 90-ft. section developed four transverse cracks, six sections, three such cracks; and twenty-one sections, two. In no case do these cracks show any appreciable wear, but they will doubtless prove to be a source of annoyance in maintenance of the road after several years' service.

In this connection, it may be of interest to state that a twelve-ft. pavement, 6,916 ft. long, was completed in September, 1912 by the county of Huron as a monolith. The aggregates were Bellevue crushed limestone and Canadian sand. Six sacks of cement were used per cubic yard of concrete. Over one-half of the road the thickness is 6 ins. and the pavement rests on clay subsoil. Over the other half the average thickness is 5 ins. and the minimum 1 ins. laid on old sandstone foundation. On inspection made last month, it was found that 85 transverse cracks had formed, ranging in spacing from 9 ft. to 159 ft. The cracks are very irregular and several are badly worn. The same general condition both as to number and spacing of the cracks existed on each half of the road.

The much discussed question of the advisability of treating the surface of concrete pavements with a bituminous mat, is being investigated very carefully by the Department. The results thus far obtained on 23 roads thus treated have rather discouraged further general application. The method of surfacing the concrete with the bitumen is as follows: After the concrete is well hardened, a period of not less than 10 days being specified shall elapse after the concrete has been placed, tar or asphalt heated to from 200° to 250° F. is applied, in quantity equal to ½ gal. per square yard, and spread uniformly over the surface. While the tar is still soft there is applied a layer of coarse sand or pea gravel in amount of 1 cu. yd. to about 100 sq. yds. of surface. This forms a mat varying in thickness from ⅛ to ¼ in. Present specifications limit the covering to tar products and gravel, ranging in size from ⅛ to ¼ in.

On inspection just made by the speaker and Division Engineer Harwood Lersch, the date obtained show that on only four out of 23 roads treated has the bitumen given good results, on all other roads it has peeled off in varying degrees. An examination of the tar mat that peels off almost universally shows that the tar has adhering to its under surface a film of inert mortar, from 1/32 to ⅛ in. in thickness, that has been separated from the surface of the concrete. It thus seems that the peeling is not due to failure of the adhesion of the tar to the concrete, but the peeling is due either to an action of some ingredient in the tar on the concrete surface, or more likely to the nature of the concrete itself, which makes it possible for the tar to pick up the top film of the concrete. Where the treatment is an oil asphalt, this condition is not true, and it is the experience of the Department that such a product cannot be successfully made to adhere to the concrete.

The life of a bituminous tar treatment (where the same adheres to the surface) depends upon many factors. To accurately determine these factors is somewhat of a problem. Examination of the various roads reveals that tar mat filled with sand wears off much faster than one filled with pea gravel, which will resist abrasion to a higher degree. The treatment of tar and sand on Road No. 11, Youngstown-New Bedford Road, indicates a probable life of not to exceed two years, same having worn down to a thin mat not exceeding 1/16 in. after a use of only seven months under heavy mixed traffic. At a cost of 9½ cts. per square yard for such treatment, which is an average cost on all roads treated, the maintenance would be rather excessive.

An interesting fact has revealed itself on Section 19 of the South High Street Experimental Road at Columbus, constructed under force account by the maintenance bureau of the Department. A Dolarway treatment was applied at the rate of 0.36 gals. per square yard, and covered with coarse sand. The 33 ft. at the south end of this section was given two applications of the tar and sand. The part given one application peeled badly, while the part given two applications holds, and is in good condition after 15 months' use under very heavy mixed traffic. The latter presents very much the appearance of an asphalt pavement. On section 20 of this same road where Tarvia was used, what peeling has taken place has been on the parts which received the least tar.

Owing to the apparent short life of the bituminous wearing course as heretofore applied, it would appear in the light of present knowledge of the subject to be uneconomical to apply such surfacing, or attempt to maintain a bituminous top on the pavements thus far treated in this manner. However, the use of tar for treating cracks which develop in the pavement, and the use of tar and mineral aggregate for patching defects such as worn joints and holes, is very effective, and apparently the cheapest method of maintaining a good surface on such a road. Such repairs should be made at least once every year. The cracks, joints, or depression should be swept clean of all foreign matter before the heated tar is applied. The mineral aggregate should be clean, thoroughly dry, and of a size practically as large as the depth of the hole. If fine material is used the traffic will drive the tar to the top. This tar concrete should be thoroughly tamped. While asphalt may be used for filling of joints and cracks, it is usually advisable to repair depressions in the surface at the same time, and for this latter purpose tar has given the best results.

There are, of course, exceptions and situations that can be cited where possibly some advantage can be gained by the use of the bituminous surface treatment over the entire pavement, as for example in the case of badly cracked and checked pavements. To date, no systematic operation of maintenance has been developed by the Department, but a complete organization for maintenance, under the immediate charge of superintendents employed by the Department, is being perfected.

The average cost of the concrete pavements completed to date is $1.128 per square yard. This cost is the contract price paid and does not include the cost and expense of engineering and inspection, but does include the cost of grading and finishing of shoulders and ditches and a small amount of accessories. Deducting from this average cost the average cost of surface treatment, which is 9½ cts. per square yard, there remains $1.033 as the average cost per square yard of plain concrete including grading, shoulders and accessories. The average cost of grading, finishing of shoulders and ditches and accessories is $0.167 per square yard of paved surface. Deducting this from $1.033 we have 86.6 cents as the average cost per square yard of the plain pavement alone exclusive of engineering and inspection costs.

Examinations of the details of the 29 roads completed, during and after construction, gives information from which we are able to draw the following conclusions:

(1) That careful and intelligent inspection and the use of only first-class materials are essential to the construction of concrete pavements to insure permanency.

(2) That the use of bank run gravel or crusher run limestone should not be permitted.

(3) That a rich mixture is necessary as a factor of safety against irregularities in the mixing and placing of concrete.

(4) That the wearing quality of the pavement is often greatly impaired by insufficient protection against too rapid drying.

(5) That there is no apparent difference in the permanency of pavements constructed on either a flat subgrade or on a subgrade having a crown.

(6) That the weakest points in the pavement are at the expansion-contraction joints and that tarred felt is preferable to a poured joint.

(7) That the part of the shoulders next to the concrete pavement should be reinforced by placing stone or gravel macadam on 1 to 2 ft. of the shoulders next to the pavement. This will avoid the usual formation of a rut in the shoulder which tends to retain the surface water.

(8) That a bituminous surface treatment is not essential to a good concrete pavement.

(9) That observations indicate there is no special value in roughening the surface of the concrete preparatory to putting on the surface treatment, and such roughening will tend to increase the wear if the surface coat is worn away and not replaced.

(10) That asphalts used as a surface treatment for concrete have peeled more than tars.

(11) That the results secured in the surface treatment of the present time do not warrant its use in great quantities, but experiments should be carried forward in sufficient number to learn further facts and establish, or disprove, those already thought to be known.

(12) That first-class concrete pavements have every indication of ability to stand up under heavy steel tire traffic.

Portable Railways for Hauling Materials for Road Construction.

(Staff Article.)

Discussion at the recent national Conference of Concrete Road Building of the committee report on methods of hauling materials for road construction included reference to the successful use of portable railways. These railways, as many of our readers know, consist of track rails and ties made up into units capable of being carried by two men, and of similar turntable, switch and other special units, all of which are laid down on the ground and connected up to form a continuous line. Transportation over these railways is accomplished by special small cars hauled either by horses or by "dinky" locomotives. For roadwork the portable track is laid along the side of the grade or along the shoulders, and extends from the railway siding, gravel pit, stone quarry or other source of supply to the places where work is being done.

The accompanying engravings show two views of portable railway or road construction and the following information concerning the method and cost of operating such a railway on road construction near Lockport, N. Y. We are indebted for this information to the Orenstein-Arthur Koppel Co., whose equipment was used.

The equipment used consisted of about four miles of narrow-gage portable track, 40 36x24-in. dump cars and two 5-ton dinky locomotives. The cars were hauled in trains of 12 cars each, the arrangement being so made that there was always one train of loaded cars on the way to the site of the work, one train of empties returning for material and one train of cars being loaded. The average amount transported was 80 cu. yds. per day. The method of loading the cars at the crusher bins is shown by Fig. 1. The construction of the portable track along the shoulder of the road is shown by Fig. 2.

While hauling stone three miles from a crusher at the quarry to the road the cost of operating the trains was as follows:

Item—	Amount.	Per cu. yd.
Materials:		
Fuel and oil for locomotives and cars	$ 8.00	$0.100
Labor:		
2 engineers at $2.75	5.50	0.069
2 brakemen at $1.75	3.50	0.044
1 track foreman at $3	3.00	0.037
1 track laborer at $1.75	1.75	0.022
Totals	$21.75	$0.272

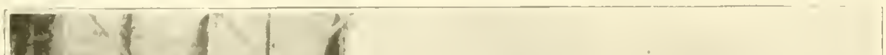


Fig. 1. Loading Narrow-Gage Dump Cars With Crushed Stone at Crusher Bin.

As the material was hauled three miles the unit cost was 9 cts. per cubic yard per mile. The average cost of grading the shoulder or berm of the road ready for track laying and laying track was between 2 and 3 cts. per foot of track.

Proposed construction work at Lyon, France and vicinity includes sewerage, $4,800; sidewalks and sewerage, $2,900; drainage and water filtration, $8,000, asphalt streets, $11,600; canal work, $5,300.

Construction of the Crackless Concrete Pavement.

Contributed by S. T. Morse, Consulting Engineer, Carlinville, Ill.

The production of the crackless concrete pavement is the thing most desired and sought by a vast number of engineers now striving toward its attainment. It is at present merely an idealistic structure, the methods to be used for constructing which are now under large discussion, claiming the attention of all specialists in the paving world.

Sufficient success has been attained here and there throughout the country to indicate the beauty and utility that will be developed in pavements of this class when at last perfection in methods of construction has been reached. The demonstrations so far made have aroused such interest and enthusiasm in the public, who do not foresee questions of maintenance so quickly as engineers, that the demand for more roads of concrete is becoming most insistent and the amount of such construction has increased at a rate beyond all precedent. Until the year 1913 the growth has about doubled each year and may be quite accurately represented by the curve $2n$ where n represents the number of years elapsed since 1890. There are a few localities, however, where, according to the records, the high-water mark has been reached and passed, such as Portland, Ore.; Kansas City, Mo.; Sioux City, Ia.; Bozeman, Mont., etc. This falling off is undoubtedly mainly due to the formation and prevalence of longitudinal cracks which have so universally appeared in concrete pavements after the first winter of service.

A concrete pavement when first laid and before unsightly cracks have formed, approaches nearer to the long-desired perfect general service pavement than any road surface yet developed. It has low tractive resistance, is not slippery, rather gritty; is sanitary; reasonably durable; should require small maintenance; is moderate in cost, and its aesthetic qualities add a suggestion of good taste and even luxury to the road, which factor is today in increasing public demand. With so much of elegance and service to be attained it becomes imperative that engineers adopt methods of construction which will insure a much higher percentage of success than is now commonly secured, lest the public will shortly come to believe that pavements of concrete are a snare and a delusion and turn squarely against them.

Transverse cracks are of comparatively small moment, but longitudinal cracks constitute the point of vulnerable attack made by the advocates of other forms of pavements, and comprise and induce at least 80 per cent of the damage appearing in the average pavement. Within the last twelve months the writer has discovered the cause of the difficult, ascertained the remedy, and with confidence recommends such pavements to his clientage and all municipalities and counties who will put away present guess methods of design and adopt the true scientific system which leaves nothing to uncertainty.

The forces which cause longitudinal cracking of a concrete roadway are now pretty generally conceded to arise from slight vertical movements which occur in the shallow foundation as the seasons change throughout the year. The assumption has heretofore been almost universally made that by means

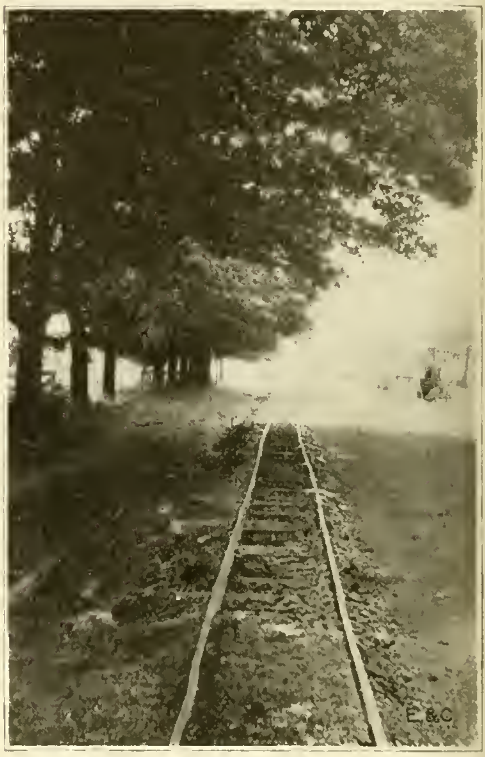


Fig. 2. View of Narrow-Gage Portable Track Laid Along the Shoulder of a Road for Construction Purposes.

of much rolling and great refinement in compacting the subsoil absolute stability in the foundation may be obtained. This is a mistake, as witnessed by the fact that our most careful constructors in Michigan, Maryland, Illinois, Iowa and other states who use extreme care in all foundation and drainage work are producing as large a percentage of longitudinal cracks as is found in the work of the average engineer. In fact more vertical movement is realized by over-compacting earth to a point beyond its natural density than by just sufficient rolling to bring it to a point of natural density. Experiments have been conducted by the writer and others demonstrating this fact. The true idea of perfect construction is rather to provide a pavement slab with sufficient strength to maintain its integrity under all probable conditions of the foundation, expecting it to freeze and swell, dry and shrink, and expand again under the action of many elements. To accomplish this requires the slab to be a self-supporting structure over a span of at least two-thirds its width.

The writer has been much interested in a mathematical demonstration showing the supporting power of plain concrete slabs, given in the ENGINEERING AND CONTRACTING of Feb. 11, 1914, by Mr. J. W. Pearl of Chicago. He clearly demonstrates the great strength to be expected in a concrete slab where circular action can be obtained; whereas most designers of concrete pavements have in the past assumed that "The strength of a plain concrete slab in acting as a beam to carry the load over a low spot or soft place in the foundation is very slight." The most surprising fact brought out by the formulas he presents is that in the larger widths of pavement, 15 ft. and over, the stresses produced by a large concentrated wheel load in comparison with the dead load weight of the pavement is relatively small. The writer has extended the development of

Mr. Pearl's equations to a form more easily applied to the concrete pavement problem and herewith presents the result for the use of those interested, as follows:

It is known that the forces causing the greatest stress act in a section taken at right angles to the center line. Considering such a section, the slab supporting the live wheel and dead loads is only partially a continuous slab, and the radius of the supporting circle may be taken to be one-half the width of slab. Letting P represent the concentrated load; L, the width of pavement in inches; t, the thickness of slab in inches; w, the weight of slab per square foot; and f the stress per square inch in extreme fibre of concrete, the equations may be combined and condensed to the form:

$$t = \sqrt{\frac{0.36P + 0.0026wL^2}{f}}$$

For a wheel load of 12,000 lbs. and a unit stress of 200 lbs. per square inch, permissible in construction of this class with a concrete of the high class now used, the following thicknesses of slab are found to be required:

Width of pavement, ft.	Thickness of slab, ins.
8	5.5
10	6.0
12	6.6
16	8.5
20	11.0

Under the showing made in the table, it is not surprising that so many pavements 15 ft. in width and over have cracked longitudinally; that any have withstood the strain is due to the fact that higher unit stresses than those herein assumed as safe are brought into action. According to the published reports, 16 per cent of the Wayne County, Michigan, pavements over 12 ft. in width have cracked longitudinally. In order that any pavement 16 ft. wide and 7 ins. thick may remain unruptured requires that the concrete develop a modulus of rupture of over 300 lbs. per square inch. In my judgment this fact pays a high compliment to the quality of workmanship secured in the above mentioned county.

Under equivalent assumptions the results given by the above formula agree almost identically with those obtained by the use of other formulæ for thicknesses hereinbefore published (ENGINEERING AND CONTRACTING, July 9, 1913). Examples of pavements built within the above requirements and which have not failed, but are today the perfect examples which it was hoped they would remain, may be mentioned as follows:

Eight miles of pavement 8 ft. wide and 6 ins. thick, constructed by the Minnesota Highway Commission; one street in Lima, O., 10 ft. wide and 9 ins. thick; all sections in Wayne County, Michigan, 12 ft. wide and 7 ins. thick; one mile at Greenville, Ill., 16 ft. wide and 8 ins. thick. A thousand instances of those which have been constructed having a width manifestly too great for their thickness may be cited.

At the recent National Conference on Concrete Road Building held in Chicago the [illegible] brought out by Leonard C. Smith of the University of Wisconsin showing the ineff[illegible]ness of steel reinforcement to prevent longitudinal cracking should show all who [illegible] heretofore placed their reliance [illegible] of safety that it is only a del[illegible] not scientifically informed. Steel [illegible]ed for use in strengthening a [illegible]ment where the first requirement [illegible] concrete shall not crack, and [illegible] the 0.1 per cent now al[illegible] valueless in such work. At [illegible]ference the committee on re[illegible] to report for the reason [illegible]ber who actually prepared [illegible] demonstrated the ineff[illegible]gance of steel reinforcem[illegible] uses that others on the c[illegible] subscribe to any such re[illegible] expended for steel would [illegible]pended in providing additional concrete.

In building a wide pave[illegible] thickness, there is only [illegible] above shown extreme but necessary thicknesses can be safely and economically avoided, viz., by the construction of a protected longitudinal joint, which has the effect of reducing the width of slab subjected to the active stresses. The form of protector plate should be one suited to the rolling action of wheel traffic, and will probably be different from the form now used for transverse joints. By this means about $3,000 per mile may be saved on construction costs, together with better and more certain results.

When built under the above requirements for thickness, we may reasonably expect to see a much nearer approach to the attainment of the crackless concrete pavement.

Methods and Cost of Constructing a Bituminous Macadam Road Across Lava Flows on the Island of Hawaii.

Contributed by M. J. Adams, Chief Engineer
Hawaii Loan Fund Commission,
Hilo, Hawaii.

A road improvement which presented some unusual features has recently been executed on the Island of Hawaii in the construction of 9.03 miles of bituminous macadam road across some of the more recent lava flows, including the latest flow of 1907.

Before improvement the roadbed consisted merely of loose lava and the conditions were such that the highest powered motor cars were sometimes stopped while trying to ascend the steep grades. Although some Pahoehoe or smooth flow was encountered the greater part was "A-A" (ah-ah) or clinker flow. It was not unusual to ruin a set of automobile tires in running the distance of 9 miles. Some 3 or 4 years previous to the construction of the improvement the county had covered a short stretch of the flow with an earth filler which had compacted and worn very well. The Hawaii Loan Fund Commission provided for the making of additional experiments with earth filler. One of the local transportation companies kindly furnished an auto truck and driver, a small force of laborers was hired and the work was carried on for two weeks to arrive at a fair approximation of the cost of applying earth filler. However, because of the fact that the only available source of supply was beyond one end of the contract, with a consequent long haul and the fact that the amount of filler in sight was limited, comparative bids showed the cost of earth filler to be very nearly the same as that of bituminous binder, so the latter was used throughout.

A short experimental section was tried where the bituminous binder was merely added to the natural material as it occurred in the road. Owing to the average large size of the stone, the rounded condition of that in the ruts, the variety of sizes, and the occasional presence of dust pockets just under the upper layer, it was not deemed advisable to continue this method. With the exception above mentioned the 9.03 miles were constructed with a 4-in. broken stone macadam top course and bituminous binder grouted.

The grading was a difficult and expensive piece of work. The greater part of the material handled was of such a nature that unless handled with due respect it would puncture the flesh. To walk over it would [illegible] up a [illegible] pair of shoes in a very short time.

The frequent [illegible] were [illegible] to be [illegible] below a depth of a few feet [illegible] the hardest kind of rock, which was [illegible] on account of the frequent blow holes and pockets of loose material. Practically all excavated material was wasted and embankment borrowed, as it was considered cheaper to do this than to move it. The ruling grades were reduced from 15 per cent to short stretches of 9 per cent. As the greater part of the road is on long tangents with corresponding lengths of sight, [illegible] travel [illegible] the width of macadam was made 10 ft.

The materials and methods used in constructing the macadam are described in the following specifications.

After the sub-grade has been brought to grade and thoroughly rolled No. 3 stone (stone passing a 2½-in. ring) shall be evenly spread to such a depth that the whole course shall have, when completed, the required thickness. Blocks of wood of proper size shall be used to gage the depth of loose stone. Care must be taken to preserve the grade and crown and prevent a wavy surface; and wherever irregularities or depressions occur, the top course must be loosened up and the No. 3 stone added to take out such irregularities and depressions.

The course shall be rolled with a self-propelled roller weighing at least 5 tons, until the surface is firm and compact, after which 1.5 gals. of bituminous material, as hereinafter specified, heated to a proper temperature, shall be evenly spread over each square yard of surface. Immediately thereafter, 0.5 cu. ft. of dry No. 2 stone (stone passing a 1¼-in. ring, but retained by a ⅝-in. ring) per square yard, shall be evenly spread upon the surface to fill the voids, and be rolled until no more can be forced into the course. All loose material shall then be swept off. Next 0.5 gal. of the bituminous material, heated to a proper temperature, shall be evenly spread over each square yard of surface; upon this shall immediately be spread ⅜ cu. ft. of dry dustless screenings for each square yard of surface and the rolling be continued until a firm, smooth surface results, conforming with the plan in longitudinal and transverse section.

This wearing surface shall be maintained and renewed if necessary until the entire work has been accepted, and if more than one renewal is required stone dust will be permitted for subsequent renewals.

It was specified that the stone used should be the best obtainable from the flow. This is a light weight lava rock with a number of small holes. Because of this the quantity of asphalt used was raised, from the 1.65 gals., originally specified, to 2.0 gals. per square yard. The asphalt used showed the following characteristics:

Homogeneous	Yes
Gravity at 77° F.	1.03
Penetration at 77° F.	6
Per cent loss on heating at 325	Trace
Per cent bitumen soluble in CS_2	99.3
76 per cent naphtha	74.0
Carbon tetrachlorides	99.4
Per cent fixed carbon	19.2
Open flash	560°
Melting point	142°F

All asphalt was poured from hand pots. For a considerable portion of the length of the road there was a high wind much of the time, rendering uniform application difficult.

Owing to the condition of the sub-grade and the fact that all fills were retained by slope walls a light weight roller was specified. During the progress of the work it was found, that, owing to the light weight of the stone and its brittleness and the condition of the sub-grade, it was possible to pass the roller only once or twice over the loose stone, before the application of the bituminous material. If more than this was done the roller worked down through and pushed the stone in waves. The course was thoroughly rolled after each application of asphalt.

The main features of the contractor's plant were: One 5-ton gasoline dump truck, one 5-ton gasoline platform truck, one light run-about with delivery body, one [illegible]-ton per day jaw crusher, later replaced by a 60-ton per day gyratory crusher, one 7-ton gasoline roller, [illegible] several [illegible] and [illegible].

The road was constructed by hauling over the finished work. The 5-ton truck was used for hauling macadam stone and the platform truck for hauling asphalt and supplies, frequently working far into the night. On the last 3 miles the contractor hauled to a point near the end of the completed work with his 5-ton truck and dumped into temporary bins, from which point the macadam stone was hauled to the sub-grade by wagons. This was because of the large tire expense when the truck was handled on the sub-grade.

All water for camp purposes and stock had to be hauled an average distance of 8 miles. The asphalt and all camp supplies had to be hauled an average distance of 18 miles from the nearest port and up a difference in elevation of 1,800 ft., the most of which occurs in a succession of long grades of 6 to 8 per cent.

The contract prices were: A little less than $1,000 per mile for grading, macadam top course $3.85 per cubic yard in place, and bituminous material $0.28 per gallon in place.

The total cost of the improvement was about $54,000 or about $1.02 per square yard of wearing surface.

The first few miles of the road stood the continuous traffic of the 3-ton truck and some traffic of the 5-ton truck for 10 months, practically without a mark. Beyond that, largely owing to the fact that there was no fine material in the loose sub-grade to help bind it, there was rutting and occasional breaking through the road crust. These places were successfully patched and beyond a slight waving, which is not objectionable, except to a short bodied car moving at high speed, the finished road is in good shape.

Payment for the construction of this road was made from funds provided by the Hawaii Loan Fund Commission, of which Mr. M. J. Adams, Hilo, Hawaii, is Chief Engineer.

Methods and Costs of Constructing a Shell Road at the United States Naval Academy, Annapolis, Md.

Several sections of road aggregating 7,052 ft. in length upon the grounds of the United States Naval Academy at Annapolis, Md., were improved between April 17, 1913, and May 16, 1913, by shaping the old road, a considerable part of which had been previously surfaced with shell, and constructing a new oyster-shell surface. The wearing quality of the old shell surface was apparently not very good. In Bulletin No. 53, issued by the U. S. Department of Agriculture, the construction of this road is described as follows:

Grading.—The land adjacent to these roads is nearly level and the soil has been made by filling with various materials, such as clay, brick, shells, etc. The maximum cut and the maximum fill were each about 0.5 ft., since the grading was essentially only a matter of correcting slight irregularities in the grade of the old roads. The hauling was done in ½-cu.-yd. carts, and the average haul for excavation was 1,000 ft. and the maximum haul 2,000 ft. Material to the extent of 1,037 cu. yds. was moved in the excavation. It was not necessary to provide any additional culverts or other drains on any part of the work.

Surfacing.—The surfacing was done in seven sections having widths varying from 15 to 20 ft., a total length of 7,052 ft. and a total surfaced area of 11,935 sq. yds. The depth to which the shells were spread varied from 2 to 13 ins. measured loose, according to the condition of the surface, and the compacted depth was about one-third the loose depth. Shells for surfacing were brought by water for a distance of about 10 miles and hauled in 25-bushel carts 800 ft. from the scow to the road. A total of 46,125 bushels of shells were required for the work of which 24,300 bushels were delivered on the road at $0.035 per bushel, while the remaining 21,825 bushels cost $0.045 delivered at the Naval Academy docks.

Equipment Used.—The equipment used consisted of one 1½-ton tandem steam roller, one 10-ton macadam roller, 1 spike-tooth harrow, 1 water wagon, one 2-horse grader, carts, picks, shovels and other small tools. The cost of labor was $1.28, and of 1-horse teams $2 per 8-hour day.

Cost.—The total cost of the work, exclusive of depreciation and fuel for the roller was $2,803.11, making the cost per square yard $0.23. The itemized unit costs of material and labor were as follows:

Item—	Unit cost per sq. yd. wearing surface.
Materials:	
24,300 bus. shells at $0.035	$0.071
21,825 bus. shells at $0.045	0.082
Total cost of materials	$0.153
Labor:	
1,037 cu. yds. excavation at 30 cts.	$0.026
Shaping subgrade	0.020
Hauling 21,825 bus. shells	0.009
Spreading shells	0.013
Rolling surface	0.013
Total cost of labor, including incidentals	$0.081
Grand total	0.0234

Method and Cost of Constructing 7,200 Ft. of Waterbound and 9,100 Ft. of Bituminous Shell Road.

The road constructed extends from Ft. Myers, Fla., and was built between May 15 and Dec. 11, 1912. The following description and cost data are compiled from Bulletin No. 53, U. S. Department of Agriculture, Bureau of Roads:

Waterbound Shell Road.—This portion, 7,200 ft. long, of the road is located within the corporate limits of Ft. Myers. It was girded to a width of 28 ft. outside to outside of shoulders and where curbs were used the width was made 36 ft. The land adjacent to the road is approximately level and the soil is sandy throughout. The shell surface replaced an old surface of the same kind which had become badly worn, and extended the entire length of the section. It was 16 ft. wide and the surfaced area was 12,800 sq. yds. The shells were spread 12 ins. deep and compacted to 6 ins. The total amount of shells used was 3,838 cu. yds. Two hundred and five feet of vitrified clay culvert pipe, ranging in diameter from 15 to 24 ins. was used. A reinforced concrete girder bridge, having a span of 24 ft., a 24-ft. roadway width and a 6-ft. height of opening, was constructed by contract for $1,540.

The equipment consisted of a 10-ton steam road roller, slat-bottom wagons, and hand tools. The wood used for fuel for the roller was cut by a county convict force and the cost is not included. The cost of labor was $1.50 per 10-hour day and of teams $5 per day. The cost was as follows:

Item.	Per sq. yd.
3,838 cu. yds. shells at $1.02	$0.306
Clearing, grading and trenching	0.043
Scarifying old road	0.009
Hauling shells at $0.309	0.093
Spreading shells at $0.087	0.026
Rolling shells at $0.04	0.012
Total	$0.491
Culverts and bridge	0.140
Grand total	$0.631

Bituminous Shell Road.—This section is 9,100 ft. long and had been partially graded before the object-lesson work was begun. The available cost data concerning grading were, therefore, not complete. The width graded was 21 ft. and the width surfaced 16 ft., making a total area of 21,267 sq. yds. graded and 16,178 sq. yds. surfaced. The shells were spread to a depth of 11 ins. in the center and 10 ins. on the sides, and compacted to 7 ins. in the center and to 5 ins. on the sides. The shells were old and rotten and ranged in size from whole oyster shells to small fragments. After this material had been spread and partially compacted, a thin bituminous binder was applied to the surface cold at the rate of ½ gal. per square yard, and left undisturbed until the next day. Shell fines and sand were then spread over the bitumen and the surface was rolled until thoroughly compacted. The length of road treated with bitumen was 8,950 ft. and the area 15,911 sq. yds.

In constructing the culverts necessary to carry the road over small streams 168 ft. of 24-in. vitrified clay pipe, 56 ft. of 60-in. half-round corrugated pipe, and 32 ft. of 18-in. circular corrugated pipe were used.

The equipment used consisted of a 10-ton steam roller, a road girder, slat-bottom wagons of 2½ cu. yds. capacity, and hand tools. Labor cost $1.50 per 10-hour day and for teams $5 per day. The cost of the section was as follows:

Grading and Wearing Surface.

Item—	Unit cost per sq. yd. wearing surface.
Materials:	
5,608 cu. yds. shells at $1.0125	$0.3510
8,089 gals. bitumen at 9 cts.	0.0458
Demurrage on car	0.0051
Wood for fuel for roller	0.0092
Total materials	$0.4111
Labor:	
Clearing, grading and ditching	$0.1112
Trimming shoulders and ditches	0.0108
Shaping subgrade, 16,178 sq. yds.	0.0006
Hauling 5,608 cu. yds. shells at $0.446	0.1545
Spreading shells at $0.0678	0.0236
Sprinkling 16,178 sq. yds.	0.0051
Rolling 16,178 sq. yds.	0.0184
Hauling bitumen	0.0043
Spreading the bitumen, 15,911 sq. yds.	0.0061
Applying shell fines and sand, 15,911 sq. yds.	0.0078
Incidentals	0.0001
Total labor	$0.3428
Total cost grading and wearing surface	0.7539
Total cost of culverts	0.0452
Total cost of road	$0.8040

Brick Roads Without Permanent Curbs or Edging.—In a paper presented at the twenty-ninth annual meeting of the Illinois Society of Engineers and Surveyors by Mr. W. T. Blackburn, Paris, Ill., the construction of two brick roads built without permanent curbs or edging was described as follows: I have in mind a brick country road 9 ft. in width that has been in constant service since 1899 (14 years). The roadway was constructed according to what is known as the two-course method and was composed of brick on edge for wearing surface, resting on a cushion of sand, supported by a number 2 paving block foundation. Two-inch boards were placed along both edges level with the finished brick surface to act as an edging. The earth was graded up level with the surface of the pavement. The boards are almost all gone, either having decayed or having been lifted by frost and removed. This roadway is in excellent condition and has demonstrated that a country road can be successfully built in this manner.

An experimental section of brick road about 300 ft. long has been recently constructed to connect the above described road with another brick pavement. This section is provided with a 4-in. concrete foundation made smooth by the use of a template drawn over boards set to grade along the sides of the foundation. On the foundation was placed a well-compacted sand cushion 1½ ins. in depth, and on this was placed a brick surface made smooth by rolling. Care was exercised that the joints were left open and they were afterwards filled with a cement filler made of 1 part cement to 1 part fine sharp sand. The precautions were taken to place the fractured ends of the bats away from the edge of the pavement, and to use nothing smaller than half brick at the edges, leaving the edge of the pavement finished with the perfect ends of blocks. The boards were then removed and the earth tamped along the edges of the pavement.

This experiment bids fair to demonstrate the practicability of building brick roads without the added expense of an edging. This pavement was built 9 ft. in width and should necessity require a greater width a 3 ft. crushed stone shoulder can be placed on each side.

Increase in Motor Traffic on Oakland, California Streets.—According to the annual report of the Commissioner of Streets, the most interesting result of an extensive traffic census taken in that city is the discovery that the motor traffic now exceeds the horse-drawn traffic by a substantial margin, a total of 69,593 motor vehicles having been counted as against 60,242 horse-drawn vehicles; that is, 53.6 per cent of all traffic passing 25 selected stations consisted of motor vehicles of all kinds, and 46.4 per cent of horse-drawn vehicles.

Engineering and Contracting

Devoted to the Economics of Civil Engineering Design
and to Methods and Cost of Construction

olume XLI. CHICAGO, ILL., MARCH 11, 1914. Number 10.

A Correction.

Through error the article, "Methods and osts of Painting an Interurban Railway Car, reen Bay, Wis.," which was published in ur issue of Jan. 14, 1914, was not credited "Electric Traction" from which the information was taken.

Engineers as Public Speakers.

The *Electrical World* comments upon the de-rability of combining a knowledge of the rt of public speaking with a knowledge of ectrical engineering. To quote:

> The work of no engineer has so great or so nportant a bearing upon the business and home fe of a community as that of the electrical ngineer. Therefore, it would seem clearly de-rable that the electrical engineer of the immeiate future be sent out into the world with not nly the necessary equipment of technical nowledge, but with at least an elementary raining in the persuasive art of public speaking, a fact that progressive colleges are now eginning to appreciate.

To all of this we heartily subscribe, if one ord be changed in each of the two sentences uoted. We believe "civil" should be substituted for "electrical," for surely no class of ngineer comes more frequently in contact ith engineering problems of public interest han the class of engineers designated as ivil."

The text that furnished the interesting elec-rical preachment of our contemporary was an ddress by a civil engineer, Mr. Henry Matson aite, city manager of Dayton, Ohio. At the ftieth anniversary of the Theta Xi fraternity n New York City, Mr. Waite spoke on "The ity as a Business Proposition," and in the ourse of his interesting address urged the aining of young engineers in the art of pub-c speaking. He pointed out that we are at e portal of a new era of engineering, and at the engineer of this new era must be creasingly in the public eye. That he may e able to communicate his ideas to the pub-c, whether the audience be a board of di-ctors or a meeting of taxpayers, it is desirable that he be able to think without confu-on and speak without halting in the presence f an assemblage.

This is not easy for a "high strung" man, nless he frequently practices speaking before ormal audiences; and probably there are few nsitive men who ever speak publicly without ome inner perturbation Only the other day n experienced and able attorney of a large ility company told the writer that he never ose to address a court or a jury without a eeling of excitement that he had to strive o hold in check. Nervousness is not, therefore, a sign of natural inability to speak in ublic Rather is it a sign of a keen sense f responsibility as well as a mark of ambi-on to do the thing in hand well Over-nxiety breeds a nervousness that often hob-es the tongue at first, but a consciousness f thorough knowledge of the subject soon ees the tongue and enables the speaker to nvince his listeners.

There is no oratory so potent as sincerity, o rhetoric so effective as simplicity, and no gic so persuading as marshalled facts. The ndamentals of the art of public speaking e, therefore, possessed by most engineers hen they speak on engineering subjects But actice must be had in this art as in every her, if greatest effectiveness is to be attained. e urge all civil engineers to join local engi-ering clubs and societies and to practice scussing engineering subjects at the formal eetings of such organizations. Why, indeed, may not the old debating society methods be adopted? It is said that many of the orators of the West, of which Bryan is perhaps the most illustrious example, learned their art in country town debating societies. While engineers will not seek to emulate some of the methods of the orators thus trained, is it not likely that engineers could greatly profit by adopting the underlying plan of the old-time debating society?

Incidentally there might spring from such debates a public interest in engineers not otherwise obtainable.

Suppose, for example, a question had arisen as to the desirability of purchasing a water works owned by a company. Suppose the local engineers' club were to announce a debate on the question, naming, say three engineers on each side. Is it not probable that the debate would be well attended by the public? Is it not certain that the local press would report the debate fully?

Engineers are now prone to "let George do it"— George being the lawyer—whenever there is to be speaking on public matters. It is amusing, in spite of the serious side, to an engineer, to listen to the ordinary lawyer or politician address an audience on any topic of an engineering nature—roads, for example. Who of us has not smiled, and yet sat ill at ease, while such a man tried to make a Patrick Henry oration on "good roads"? Yet, if we engineers will not speak to the public, perhaps it is well that some one should speak, even if the speech be ridiculous in conception and ornate in delivery.

Every engineering college should teach public speaking and should give its students much practice in the art. There can be no serious discord on this point. But we who have graduated without such training have had to teach ourselves, as best we could, how to talk in public. This self teaching, after all, would not be so difficult if only it were possible to get sufficient practice. And it is with the idea of securing practice that we have suggested the debating society.

Larger debating clubs of engineers might profitably employ an instructor in elocution. But even if no outside instructor were hired, one or more members of the club should "coach" the speakers. Few men are good originators but few are poor critics. Our corneal lenses are wonderful microscopes for seeing faults the faults of others We need judicious fault finding to hasten the elimination of objectionable methods Fortunately the least able of us can perceive faults in the most able, but unfortunately the most able seldom hear the criticism.

In a properly conducted debating society it is the function of some one to point out the things that an erring speaker should do and especially the things he should not do.

As a youth the writer was a member of a debating society that solved all, or nearly all, the political problems that had baffled American and indeed the world's statesmen. It is true that even yet some of the most noteworthy decisions of that society have not been written into statutes. But it is equally true that several members of that society reached high political positions, largely by the "political pull" of a rugged "nerve" and strong vocal cords, aided by a bit more than the average of intellectual ability.

Politics, however, is not the aim of many engineers, when politics means divorce from engineering. But the majority of civil engineers are in politics to the extent that they are doing public work. Even railways and other public service companies are fast becoming parts of the great social machine that is under the control of public servants. We are entering a new engineering era, and with the change there will come increasingly greater opportunities for the civil engineer in public service. Indeed, may we not witness—sooner than perhaps seems now credible—the election of many engineers as chief magistrates of cities and states, with ultimately a return to first principles, namely, the election of a civil engineer to the presidency of the United States?

Providing in Advance for Additions to the Heights of Buildings.

The increasing value of real estate and the changing requirements of tenants have caused a constant demolition and replacement of commercial buildings in the business districts of our principal cities. In most cases the rental required to make the investment a profitable one has resulted in the construction of higher buildings, but in some cases an entirely different type of building has replaced the old one. There is an increasing tendency to design and construct commercial buildings to care for the present requirements only, and to make provisions for increases in their heights as conditions warrant such changes. In some cases the problem is a comparatively simple one, but in others it is extremely difficult to plan for the future. The architectural and engineering problems are usually quite simple, provided a stable foundation material is readily accessible. In Chicago and New York, where most of the important buildings are founded on bedrock, it is not, as a rule, a very costly undertaking to design and construct a building of medium height which will contain provisions for the future addition of a considerable number of stories. By slightly increasing the size of the foundation and columns, and by using the same construction for the present roof as for a typical floor, the slope required for drainage being obtained by a temporary fill and roofing surface, the problem can be solved at a relatively small additional cost If the subsoil and the height of the building are such as to require spread or floating foundation the problem may be somewhat more complicated.

Even though the engineering problems can be easily and cheaply solved, other considerations often make a wise solution exceedingly difficult and uncertain. Such varying factors as the future growth of the city, the direction in which expansion is likely to take place, the possibility of the passage of city ordinances which will materially change the height limits and the minimum size of open courts, etc., the probability of a change in the character of a district from an office to a retail, wholesale or loft section are all factors which must be carefully considered if one is to avoid great wastes. The extra cost of construction which must be capitalized for an uncertain term of years, the loss of revenue which must result if the building is demolished and replaced, and the difficulty of again attracting the old tenants and of obtaining new ones also enter into the solution of the problem. The financing of the project is always a vital factor.

We have only to make a brief investigation of the conditions as they exist in some of our leading cities to realize that little serious study has been given in the past to the problem. In some cases the architect has been able to impress upon those in authority the importance of securing an aesthetic design for their structure or of conveying the impression of great wealth and exclusiveness. We might mention some bank buildings and a few railway passenger terminals as examples of this type of building. In some respects, however, conditions are changing, due in part to a change in public sentiment as to the purposes for which funds are to be applied. Stockholders and bondholders are beginning to realize that such structures make first-class monuments and landmarks, but have a tendency to reduce dividends. We now have a number of bank buildings in which provisions have been made for the addition of a considerable number of stories whenever conditions warrant such changes. We also have, in one of our leading cities, two great railway passenger terminals; in one, considerable rentable office space has been provided—in the other there are scarcely a sufficient number of offices for the use of the officials. In one terminal provision has been made for the addition of about twenty stories to the height of a portion of it, resulting in a structure which ultimately should be at least self-supporting—in the other, little if any consideration has been given to its earning capacity, and the terminal will always be a serious drain on the resources of the railway company.

The Purchasing of Cement for State Aid Roads by Highway Commissions.

The Illinois State Highway Commission has decided to purchase all of the cement which is to be used in the construction of its state aid roads. The commission plans to furnish the cement to the various contractors and to deduct its cost from their bids. The plan is not a new one, as a similar procedure has been followed in a few states, but it is nevertheless to be commended. The construction of concrete and brick roads is still in its infancy, and for the first few years at least it will undoubtedly need constant supervision and efficient regulation if great wastes are to be avoided. It can hardly be expected that those in charge of the construction of state aid roads in the various counties will be able to get the best results at the start. The work will be new to many of them, and anything which the commission may do toward assuming some of their responsibilities should result in the construction of better roads.

The purchasing of the cement by the commission will not only mean that a "certified" cement will be used, but it will also tend to develop competition as it will give the small contractor a better chance; he will not be placed at a disadvantage simply because he cannot purchase in large quantities. It should insure a better quality of cement and better handling and storage facilities, as cement manufacturers then can hardly afford to run any risk of having their product rejected by the commission. The aggregate can often be obtained locally, and in some localities the contractor may be in a position to buy it more cheaply than could the commission. In order to buy efficiently the latter would necessarily need to buy in large quantities, and this would often result in shipping the aggregate considerable distances, thus resulting in heavy freight charges. The importance of securing good aggregate is so great, however, that an ultimate saving and better construction might result if a certified aggregate were also furnished to the contractor. As a compromise the commission might make arrangements whereby it could furnish a good quality of aggregate at a fixed price if requested to do so by the contractor. In purchasing materials the contractor acts merely as a broker, and he is usually better qualified to construct roads than to buy materials. Only the larger companies can afford to maintain efficient purchasing departments.

Road Days and Road Data.

Two short contributions published in our roads section this week came to us in one mail. Both contributions also came from western Missouri counties barely a hundred miles apart and both present thoughts well worth attention.

The contribution telling of one incident of Missouri's "road days" of 1913 may perhaps as well be noticed first. This contribution is a plea for, and in a smaller way, a presentation of evidence of the utility of "road days." Missouri's "road days" of last August were a featured newspaper item at the time and, we doubt not, most road engineers are familiar with their origin and object, and perhaps in some measure with their results. Any who are not informed may become so far enough for present need by reading the article in this issue.

Missouri's "road days" and those of Arkansas, and those, perhaps, of one or two other states, which we cannot name out of memory, are sublimations of the "road days" universal twenty-five years ago and yet common when neighboring farmers gathered and "worked out" their road taxes by improving some contiguous stony or swampy stretch of highway. The method was a modification of the bee system of more primitive days and as the writer recalls these road making bees they were but slightly less a rural social occurrence than were the quilting bees and husking bees of more familiar knowledge.

. Because of their sublimation the state "road days" of Missouri and Arkansas had publicity value which their commonplace prototypes largely lacked. They fixed thought actively for a day, or a few days, of a great number of people on the subject of road making. They temporarily brought many of these people actively into touch with road making tasks. It is impossible to tell how many minds were by these facts made more receptive to argument for improved roads, or how far those facts exerted and concentrated popular sentiment favoring road improvement, but it is reasonable to believe that there may have resulted great gains in both directions. Indeed, we can believe that the Missouri Governor's proclamation of State "road days" had full justification in its publicity value alone.

But as a method of actual construction the road making bee is not often efficient. The reasons are many, but, to name the most obvious, we have: (1) The working force is not trained in its task; it is unwieldy and cannot be effectively handled; it is not serious it is playing at a game and not performing a task for a living; (2) The work lacks co-ordination, time is wasted by want of a studied plan of construction, by lack of necessary materials provided for ready use, by need of labor saving road making machinery and tools One, we repeat, can name many reasons, but those given are enough for the purpose.

Referring specifically to the "road days" road making at Tarkio as described by our contributor: Consider the $7,000 in labor and money contributed last year for road improvements. One can admire the aroused local interest which made this $7,000 expenditure possible, and if this interest is the result of sentiment inspired by "road days" proclamations one can thoroughly approve of the method of inspiration. This is far from concluding that the $7,000 which was expended produced all in the way of road improvements that such a sum of money was capable of producing Perhaps it did in this specific instance, but the chances are greater that it did not, and the reasons are those that we have named. As a means of arousing sentiment favoring expenditures for road improvement the proclamation of state "road days" can be justified. When however, it comes to the actual spending of the money subscribed for road construction work a road building bee is as a general rule astonishingly wasteful.

The author of the second of our contributions has a notion quite different from proclamation of "road days" for bringing conviction of the value of road improvements. This notion is that figures demonstrating the value of improved roads should be collected and put before the people, who most use the highways —before the farmers and villagers. The argument is stated in the article itself and will not be repeated, but it suggests a turn of thought which has interest.

There is a multitude of statistics designed to demonstrate the value of good roads. None of these sets of figures is of the kind that carries conviction to the persons who need most to be convinced—to the resident user of roads Now the reason of this fact is worth seeking and if we seek we find that in some cases the figures are too general and in other cases while specific, represent extraordinary conditions. The resident user of roads regards the first kind of statistics as of little direct bearing in his own case and the second kind he knows does not represent conditions as he has learned them in his own community.

The thought is clear then, and we believe that this is at bottom the thought of our contributor, that the statistics needed are records of specific cases. To be convincing these records must include a wide variety of conditions and be so authenticated as to be considered truthful. It does not serve to present to the resident user of roads a nation wide generalization of losses due to poor roads or gains due to good roads, nor does it serve to quote him specific examples which he knows represent other conditions than his.

General

Comment on "Initiative—Applied and Misapplied."

To the Editors: I have just finished reading the editorial in the Jan. 21, 1914 issue of Engineering and Contracting, entitled "Initiative—Applied and Misapplied," and as I feel that the word *initiative* is a useful one and that it is misrepresented in the editorial I venture to present herewith a gentle protest and an explanation.

It is unfortunate that a large part of the words used in human speech carry more than a single meaning; naturally this leads to misunderstanding. I would accept the view, that the writer of this editorial and myself differed merely in a matter of definition and would not take the trouble to express my opinions, but for the feeling I have that my definition of the word initiative is one that is more commonly accepted in this particular application of the word than is the definition upon which the article is based, and the further feeling that students and young engineers who accept the common definition may be confused and perhaps harmed by some of the statements contained in the editorial if those statements go unchallenged. The writer of the editorial uses *initiative* and *inspiration* in a way that makes them practically synonymous, probably basing his connection of the two terms upon that definition in Webster that explains initiative as "Energy or aptitude displayed in the initiation of action, especially o action that tends to develop or open out nev fields." If this were the only definition o the word, the editorial would be less open t attack. However, the above definition of in tiative is followed immediately by this on "self-reliant enterprise," and this is the meaning that I have in my own mind when I us the expression, and, from numerous conversations I have had upon the subject, I believ it to be the meaning that is usually intende when the word is applied to the young engineer.

There is a great difference between *runnin a bluff* and having *initiative*. It would be egrettable thing should any young engineer r student after reading the article decide hat there might be a chance that any exhibiion of self-reliance on his part would be misaken for bluff and that it would be unwise for im to exhibit initiative until he had experince. There is no age limit upon initiative. elf-reliant enterprise is plainly evidenced by he sturdy two-year-old lad I pass each day, egotiating the difficult ascent from gutter to idewalk with a heavy go-cart and asking help rom no one. If that youngster follows in the ootsteps of his father and lives to be an enineer, no one will have to complain of his ack of initiative, nor will he have to wait ntil the gray flecks his temples before he ares to show that he possesses the quality.

The editor expresses his belief that the oung engineer cannot possess the *proper* kind f initiative until he has acquainted himself vith the main facts relating to the problems t hand. That statement is as illogical as the ltimatum of the father who forbade his son o go into the water until he had learned to wim. The young engineer and the old one oo must have initiative, or the ability to make start, before he can obtain possession of the acts with which he is to become acquainted. He must have the ability to take the problem hat is given him and without running to his hief for help at every turn must be able to ather his data and obtain some sort of a olution. The solution obtained by the inexerienced man probably will not be the best ossible; but it will furnish a basis for disussion between him and his chief, from which better solution will result.

A young engineering student was spending is summer vacation working in the drafting ffice of a bridge company. One day his imediate superior expressed his approval of e young fellow's work thus: "You are all ight. When I give you a job you always ut something down on paper for me to look t." There were young graduate engineers in e same office, men of more experience but ss initiative, who never had the satisfaction f hearing such praise applied to themselves: hen a job was handed to them they sat round until someone told them what to do rst. They were able to go through the mehanical operations of drawing and lettering; ut a thinking role was too heavy for them. A arge part of the lack of initiative is plain aziness.

While I was engaged in railroad work I ut an engineer at the task of consolidating number of maps. He was a man about forty ears old and with a fair amount of experince behind him; but was quite without the bility to help himself. Throughout the time at he was working upon that drawing he ade regular and frequent trips to my desk whine for help. I use the word *whine* beuse it describes accurately his tone of voice. feel that I shall be able, always, to recall that ne and the words, "I can't make the azimuth eck." His manner implied that I was reonsible for getting him into his difficulties nd that it was therefore my duty to get him ut. In reality there was no difficulty that he ould not have overcome by his own efforts.

The student need not delude himself with e idea that he can go through his college urse without practicing self-reliance and en suddenly, upon graduation, blossom out to a man of initiative. The student who exects the instructor to point out the page in e text-book where he will find the informaon necessary for the solution of his probm will, later, expect his chief to think out l the details of his work for him. Such men ttle to the bottom surely and rapidly. Those ho are responsible for the student's training, alize this and try to impress upon him the eat value of self-reliance. We do not like see our work counteracted by such writings the article in question. I know from obrvation, that, to the young man who has t yet learned to weigh all statements, printl or otherwise, in his own mind the printed atement has an authority not at all in proortion to its real worth and quite outranks e spoken advice he may receive.

I take issue with the editorial further, in regard to the propriety or desirability of the employer raising the question of initiative at the time he engages the services of a young engineer. If the young man has that quality the employer need not bother as to whether or not he will continue his studies. That continuance is a foregone conclusion: every problem the young engineer handles will call for study and investigation and if he be self-reliant he will do that studying and make those investigations for himself. If he has no initiative he will expect the man above to supply the starting power. He will deceive himself with false reasoning: "The boss knows how this should be done, therefore why should I worry and grow short-sighted looking this matter up in some book. Verily, I will let him help me out; doubtless he will be overjoyed to see how I bow before his superior knowledge and I shall grow great in his eyes."

There may be employers who like the man without initiative but there never will be enough to supply the demand.

Very truly yours,

L. F. Van Hagen.

Assistant Professor of Railway Engineering,
Madison, Wis., Jan. 21, 1914.

(The above letter was received as a "gentle" protest to our editorial "Initiative—Applied and Misapplied." Evidently the difference of opinion is due to different meanings of the word "initiative" in the minds of the writers of the editorial and the letter. Neither derivation nor usage warrants the assumption of our critic that the accepted definition of "initiative" is "self-reliant enterprise," instead of "ability to originate." Even the writer unconsciously follows the latter meaning, for he says "The young engineer and the old one too must have initiative, or the *ability* to *make a start*. The writer of the letter often uses the word "self-reliance" and gives it the same meaning as "initiative." Initiative is allied to inventiveness and many self-reliant persons may be without this faculty. We do not care to discuss in detail the examples given by our critic, as most of them are foreign to a criticism of our editorial, due to the writer's misconception of terms. The writer of the letter states, "We do not like to see our work counteracted by such writings as the article in question." In answer to this statement we reply that we should not consider it necessary to make the above comments were it not for the fact that our critic is an assistant professor in one of our well-known universities and therefore has something to do with the training of young engineers.—Editors).

Comparative Operating Costs of Steam Shovels and Electric Shovels.

There is stated to be at present from 12 to 18 electric shovels in use in the United States. From a paper by H. W. Rogers in the February, 1914, Bulletin of the American Institute of Mining Engineers we condense the following comparison of operating costs of steam shovels and electric shovels:

Consider a 120-ton shovel which is ordinarily equipped with a 5-cu.-yd. dipper and has an average capacity of approximately 2,500 cu. yds. per 10-hour day. This capacity is based on an average working time of 55 per cent and an average dipper capacity of 3¾ cu. yds. in 75 per cent. With a good grade of coal the steam shovel will require approximately 3¾ tons per 8-hour shift and will make an average of two complete cycles per minute. For the purpose of comparison, however, the maximum capacity of the shovel is taken, i. e., three cycles per minute. Under these conditions either the steam or the electric shovel will have a total working time during one shift of 8 × 60 × 0.55 = 264 mins., during which time it will make 264 × 3 = 792 complete cycles and will handle 792 × 3¾ = 2,970 cu. yds. of material.

The direct-current shovel would be equipped with two 80-HP., 500 r.p.m., 230-volt series motors on the hoist, one 40-h.p., 550 r.p.m., 230-volt series motor on the swing, one 60-HP., 550 r.p.m., 230-volt series motor on the thrust, and one 150-kw., 900 r.p.m., 250-volt direct-current generator direct connected to a 225-HP., 900 r.p.m., 2,200-volt induction motor, with four-point reversible automatic control on each motor.

The estimated power consumption during each cycle will be as follows:

	Kw.-seconds.
Hoisting	1,379
Swinging	522
Crowding	547
Total	2,448 = 0.68 kw.-hr.

Now 792 × 0.68 = 539 kw.-hr. input to the motors per 8-hour shift, or, taking into account the efficiency of the motor-generator set, 657 kw.-hr. per 8-hour shift.

As the shovel is working only 55 per cent of the time, the motor-generator set will be running light 45 per cent of the time, or 8 × 60 × 0.45 = 216 minutes.

The power consumption on the set when running light will be approximately 16.77 kw.

$$\frac{216 \times 16.77}{60} = 60.4$$ kw.-hr. loss per 8-hour shift.

657 + 60.4 = 717.4 kw.-hr. total power consumption per 8-hour shift when working under the maximum cycle.

$$\frac{717.4}{2{,}970} = 0.241$$ kw.-hr per cubic yard excavated.

The alternating-current shovel would be equipped with two 150-HP., 450 r.p.m., 440-volt motors on the hoist, one 50-HP., 720 r.p.m., 440-volt motor on the swing, one 75-HP., 600 r. p. m., 440-volt motor on the thrust, and three 125 kilovolt-ampere, 2,200–480-volt transformers, with five-point reversible automatic control on each motor.

The estimated power consumption during each cycle will be as follows:

	Kw.-seconds.
Hoisting	2,040
Swinging	759
Crowding	750
Total	3,549 = 0.987 kw.-hr.

Now 792 × 0.987 = 782 kw.-hr. input to the motors per 8-hour shift, or, taking into account the efficiency of the transformers, 796 kw.-hr. per 8-hour shift.

The no-load losses on the transformers will be approximately

$$\frac{216 \times 3.6}{60} = 13.0$$ kw.-hr. loss per 8-hour shift.

796 + 13 = 809.0 kw.-hr. total per 8-hour shift.

$$\frac{809}{2{,}970} = 0.273$$ kw.-hr. per cubic yard excavated.

Labor per shift	Steam.	Electric.
Shovel runner	$ 6.00	$ 6.00
Craneman	4.00	4.00
Fireman	2.50	....
Six pitmen at $1.75	10.50	10.50
One watchman	1.75	....
One coal passer	1.50	....
Teaming (½ day)	2.50	....
Oil and waste	1.50	0.75
Total	$30.25	$21.25
Saving, electric over steam	21.25	
Per shift	$ 9.00	

For convenience in comparing the costs of operation on steam and electric shovels the costs are all reduced to a day basis.

		Electric	
	Steam.	Direct current.	Equivalent alternating current
Interest at 6 pct.	$ 5.20	$ 7.75	$10.85
Depreciation at 4½ pct.	4.03	6.00	8.43
Repairs at 10 pct.	8.66		....
Repairs at 6 pct.		7.75	10.85
Labor, per shift	30.25	21.25	21.25
Total cost per shift	$48.14	$42.75	$51.38

It has been assumed that, owing to weather conditions, delays, etc., the shovel working year consists of 150 days and the above figures are based on this assumption; also that the shovel is only working one shift a day.

If the shovel works three shifts a day instead of one shift a day, the interest and depreciation will remain the same, provided the shovel is kept in repair. It is reasonable to assume that the repairs will increase when working three shifts, but not in direct proportion; therefore this item has been increased 50 per cent.

	Steam.	Electric—Direct current.	Electric—Equivalent alternating current.
Interest at 6 pct.	$ 5.20	$ 7.75	$10.85
Depreciation at 4⅔ pct.	4.03	6.00	8.43
Repairs at 15 pct.	13.00	....	....
Repairs at 9 pct.	....	11.63	16.28
Labor (three shifts)	90.75	63.75	63.75
Total cost (3 shifts)	$112.98	$89.13	$99.31

Disregarding the cost of coal and electric power, the saving of the direct-current shovel over the steam shovel would be $810 per year for one-shift operation and $3,580 per year for three-shift operation.

The alternating-current shovel when working one shift a day would show a loss of $486 per year. On the other hand, if this shovel worked three shifts a day it would show a saving of $2,050.50 per year. Any greater saving than that shown would, of necessity, depend upon the comparative cost of coal and electric power, but as this is variable it can only be shown by means of a curve.

Disregarding the cost of fuel, it is evident that the electric shovel is a better proposition than the steam shovel; that the direct-current equipment is far superior to the alternating-current equipment; and that the saving in operating expense will warrant the increased investment.

Comparison of Estimated with Actual Unit Prices on the Panama Canal.

To the Editors: Recurring to your valuable paper on the Panama Canal in the issue of Jan. 7, when I wrote you on that date I forgot to call your attention to an error in the statement concerning the changing of the dimensions of the locks. The dimensions recommended by the minority of the International Board of Consulting Engineers were: Usable length, 900 ft.; usable width, 95 ft. (see page 83 of the report of 1906).

Col. Goethals is quoted in your article as saying that the unit prices used in the report of the International Board of Consulting Engineers were those recommended by the Commission of 1901. This is slightly in error. Mr. William Barclay Parsons, Mr. J. W. Welcker and myself were the committee on unit prices for the report of 1906. (See page 419, Appendix R.) I submit herewith comparative schedules of the 1901 and 1906 unit prices so far as they are comparable:

COMPARISON OF UNIT PRICES, 1901 AND 1905 ESTIMATES.

	1901, per cu. yd.	1905, per cu. yd.
Hard rock excavation	$ 1.15	$ 1.15
Soft rock excavation	0.80	1.00
Earth, dry excavation	0.45	0.40
Earth, dredged	0.20	0.15
Earth, hydraulic excavation	....	0.25
Earth, hydraulic, with cutters	....	0.70
Rock excavation, under water	4.75	2.50
Coral rock excavation	....	1.50
Rock, drilled and blasted dry, removed by dredge	....	1.50
Culebra Cut:		
Excavation, all material above +10.0	....	0.80
Excavation, all material below +10.0	....	1.25
Embankments and backfilling	0.60	....
Rock in jetty construction	2.50	....
Concrete, in place	8.00	8.00
Granite, finished	60.00	81.00
Granite copings	....	54.00
Brick culvert linings	15.00	15.00
Timber:		
In mitre sills, per 1,000 ft. B. M.	100.00	100.00
In permanent cribs	....	55.00
Bearing piles, per ft.	0.50	0.55
Single track railroad, complete, per mile	75,000	....

TABLE I.—PANAMA CANAL—COMPARISON OF COSTS. 1905 ESTIMATES AND CONSTRUCTION STATEMENT TO JUNE 30, 1913.

(Construction Expenditures to June 30, 1913, from Canal Record, Vol. 6.)

	Cu. yds.	Total cost.	Unit cost.
Dry Excavation—			
Atlantic Division	2,181,998	$1,468,700.11	$0.6731
Gatun Locks	4,660,455	3,148,830.90	0.6757
Central Division	107,139,181	83,802,494.10	0:7822
Pacific Division	4,513,269	3,325,393.01	0.7368
Pedro Miguel Locks	1,133,280	1,031,422.86	0.9101
Total	119,628,183	$92,776,840.98	$0.7755
Wet Excavation—			
Atlantic Division	35,326,363	$8,422,057.42	$0.2384
Gatun Locks	38,425	20,041.19	0.5216
Central Division	....	....	......
Pacific Division	36,793,215	9,383,834.83	0.2550
Pedro Miguel Locks	....	....	.....
Total	72,158,003	$17,825,933.44	$0.2470 (average)
Hydraulic Excavation—			
Atlantic Division	29,605	$ 11,671.32	$0.3942
Central Division	57,274	10,511.46	0.1835
Pacific Division	1,549,904	1,120,907.35	0.7232
Total	1,636,783	$ 1,113,090.13	$0.6983 (average)
Total wet excavation	72,158,003	17,825,933.44	......
Total all wet excavation	73,794,786	$18,969,023.57	$0.2574 (average)
Masonry—			
Atlantic Division	224,132	$1,786,587.02	$7.9711
Gatun Dam	2,011,283	15,206,667.31	7.4496
Central Division	1,271	8,353.66	6.5725
Pacific Division	1,567	8,716.96	5.5628
Pacific Division	906,293	5,328,960.40	5.8800
Pacific Division	4,403	21,408.95	4.8624
Pacific Division	64,142	400,912.45	6.2504
Pacific Division	1,476,895	7,886,115.11	5.3397
Total	1,719,986	$30,647,731.75	$6.4931 (average)

While on this subject, it may interest you, as it did me, to know how the unit prices used by the minority compare with the actual cost of the work. I consulted the detailed report of yardage and cost given in the Canal Record for the period ending June 30, 1913. found that the unit prices were on cubic *yar* *of excavation* with no classification or distin tion other than dry and wet excavation. Ther fore to arrive at some basis of comparison, proceeded to take the total cost of excavatio wet and dry, reported at that date (June 3 1913), and divide it by the total yardage ar found the average price to be $0.5777. (F details see Table I.)

Taking the total cost of dry excavation ar dividing it by the total yardage, the avera unit price is $0.7755, and taking the total c of wet excavation and dividing it by the to yardage, the average cost is $0.2574.

TABLE II.—PANAMA CANAL—COMPARISON OF COSTS. 1905 ESTIMATES AND CONSTRU TION STATEMENT TO JUNE 30, 1913.

(Estimate of 1905.)

	Cu. yds.	Price.	All excavation.	Dry excavation.	Wet ex cavatic
Earth dredging	8,300,000	$0.15	$1,245,000	....	$1,245,0
Earth dredging	10,000,000	0.25	2,500,000	....	2,500,0
Earth dredging	2,030,000	0.70	1,421,000	....	1,421,0
Earth excavation	920,000	0.40	368,000	$ 368,000	....
Earth excavation	2,740,000	0.70	1,918,000	1,918,000	....
Earth excavation	1,100,000	0.40	440,000	440,000	....
Earth excavation	480,000	0.70	336,000	336,000	....
Earth excavation	600,000	0.40	240,000	240,000	....
Earth excavation	130,000	0.70	91,000	91,000	....
Earth dredging	12,960,000	0.25	3,240,000	....	3,240,
Rock excavation	1,160,000	1.15	1,334,000	1,334,000	....
Earth excavation	7,280,000	0.80	5,824,000	5,824,000	....
Earth excavation	39,650,000	0.80	31,720,000	31,720,000	....
Earth excavation	6,835,000	0.80	5,468,000	5,468,000	....
Earth excavation	650,000	0.40	260,000	260,000	....
Earth excavation	520,000	1.15	598,000	598,000	....
Earth dredging	1,460,000	0.25	365,000	....	365,
Earth dredging	220,000	0.25	55,000	....	55,
Earth excavation	480,000	0.40	192,000	192,000	....
Rock excavation	950,000	1.50	1,425,000	1,425,000	....
Earth excavation	950,000	0.50	475,000	475,000	....
Earth dredging	4,760,000	0.15	714,000	....	714,
Rock excavation	200,000	1.50	300,000	....	300,
Rock excavation	370,000	2.50	925,000	....	925,
Totals	104,745,000		$61,454,000	$50,689,000	$10,765,
Total dry, cu. yds.	64,445,000				
Total wet, cu. yds.	40,300,000				
Average price per cu. yd.			$0.5865	$0.7865	$0.2

Then I took our total dry excavation (es mated) and divided our total cost by it to the average per cubic yard of all classes excavation. This gave as a result $0.5865 cubic yard. Then I took the total cost of of the dry excavation and divided it by total dry yardage and the resulting figure $0.7865 per cubic yard. Then I took the tc cost of wet excavation and divided it by total wet yardage and the resulting figure $0.2671 per cubic yard. (For details see Ta II.)

The International Board of Consulting gineers adopted $8 per cubic yard for concr masonry. The average cost of the maso reported June 30, 1913, was $6.4931. It is teresting to note that the average cost of masonry on the Atlantic Division was $7.5 and on the Pacific Division $5.5628, show an economy of $1.9412 on the Pacific Divisi I account to myself for this difference three favorable elements. The Pacific D sion masonry was closer to its supply of s taken from the Chagres River at Mateo and to its supply of crushed stone from Ancon quarry, which reduced the transpo tion charge, and to the superiority of the c tilever equipment used on these locks o the cableway used at Gatun. My observat dating back to my experiences on the Chic Sanitary and Ship Canal, convinces me t within its narrower range of application, cantilever crane is far more efficient than cableway. The cableway, however, is m more elastic and has a far wider range usefulness than the cantilever crane.

Yours very truly,
ISHAM RANDOLPH,
Consulting Enginee

Commercial National Bank Building, C cago, Ill., Feb. 25, 1914.

Time and Cost Studies of Electric Drills at the Kensico Dam, Catskill Aqueduct, New York.

To secure crushed and quarry stone constructing the Kensico Dam a quarry opened up about one-half mile from the d. From the dam a service track runs to quarry and there separates into several branches cut into various parts of the quarry. Rows of holes all bottoming at the same level are drilled parallel to the tracks and they are blasted in numbers to loosen large areas of rocks. The rock is a clean and practically flawless gneiss having a tendency toward granite; in some places it is seamy enough to cause difficulty in drilling, but in the main it is hard and solid. It was desired to determine the cost and speed of drilling operations at this quarry and also in excavating footings for the dam and the Construction Service Co., New York City, undertook the necessary studies. The results are summarized in Bulletin, American Institute of Mining Engineers, for February, 1914, by W. L. Saunders, and from this bulletin we abstract the following data:

TABLE I.—TIME STUDY OF DRILLING PERFORMANCE OF ELECTRIC-AIR TRIPOD DRILLS AT THE QUARRY.

(Observations recorded in minutes and seconds.)

	No. of obs.	Minimum. M. S.	Average, M. S.	Maximum, M. S.	Actual time. M. S.	Consumed time % of total time.
Drill cutting	16	5-40	14-18	23-28	228-54	51.1
Raising drill	15	0-05	1-05	1-59	16-13	3.6
Loosening chuck (a)	7	0-02	0-11	0-45	1-16	0.3
(b)	3	1-06	1-27	1-46	(4-20)c	...
Removing bit	12	0-03	0-32	1-54	6-26	1.4
Balling hole	11	0-45	1-23	2-00	15-10	3.4
Putting bit in hole	12	0-10	0-22	0-55	4-20	1.0
Inserting bit in chuck	16	0-10	0-23	0-40	6-12	1.4
Tightening chuck (a)	6	0-05	0-10	0-20	1-00	0.2
(b)	10	0-38	0-53	1-20	8-46	2.0
Getting started	17	0-00	1-01	6-23	17-13	3.8
Cycle totals	..	8-44	21-45	41-30	305-30	68.2
Shifting drill	4d	35-32	52-00	60-00	73-18e	16.4
Miscellaneous delays	11	0-30	6-17	25-40	69-05	15.4
Total	..	.....	.....	.....	447-53	100.0

Linear feet drilled, 31 ft.; average depth of holes, 22 ft., total working time, 7 hr., 27 min., 53 sec. (a) Sleeve chuck. (b) Bolted chuck. (c) This figure is not included in "Cycle Total," for this operation was performed by one man at the same time that the other man was raising the drill. (d) During one observation a nearby derrick assisted in moving the drill, saving several minutes. (e) Inasmuch as a shift was made after every 22-ft. hole, the time for shifting properly chargeable is 31/22 × 52 = 73 — 18. Several obstructions were taken simultaneously on different drills to ascertain a fair average time required to shift before deciding upon 52 min. as the average.

The drills observed were Temple-Ingersoll, Type 5-F. The depth of hole that can be drilled without change of bit is 30 ins., but during observation the bits were usually changed after about 25 ins. had been drilled. The strokes per minute were about 400 at full speed.

The drills were operated and handled by the drill runner and one helper and moving was accomplished by hand. The method of dismantling is about the same as with the ordinary tripod drill except that in the case of the electric-air drills there is the added weight of the mounted motor and pulsator to be moved.

The diameter of the starting bits was 3½ to 4 ins. and decreased ⅛ in. for each succeeding length of steel down to 1¾ ins. for the deepest holes. The steels varied from 2 ft. 6 ins. to 28 ft. 8 ins. The steels were octagonal in section, from 1¼ to 1½ ins., with square cross bits. The shorter steels were handled by one man and the longer ones by the driller and helper, sometimes with the aid of a hook.

Two forms of chuck were observed, one a bolted chuck, and the other consisting of a sleeve sliding on a tapered key and bearing directly on the drill steel. The time required to insert the steel in the chuck and to tighten the chuck was considerably greater in the case of the former type than in that of the sleeve type, as noted in the time study made at the quarry. Both types appeared to be equally efficient.

The spacing of holes at the quarry varied from 15 x 12 ft. to 20 x 20 ft. The holes were all vertical and varied in depth from 10 to 26 ft. The number of holes shot at the quarry depended upon the quantity of rock needed in the construction work. Generally, the method employed for cleaning the holes was to pour water into them and bail out with sand pumps when changing steels.

The sharpening shed at the quarry was situated some distance from many of the drills in operation, hence there were occasional delays in receiving sharpened steels, during which time some of the drills were idle. The steels were sharpened by hand at the dam and by a Leyner drill-sharpening machine at the quarry.

The smith's work consisted of sharpening the drills and also included other repair work. He estimated that 75 per cent of his time was devoted to the drill steels. The estimate of coal burned by the smith was 500 lbs. per day.

Sticks of 60 per cent Dupont dynamite, 1½ ins. in diameter by about 8 ins. in length, weighing 12 ozs., were used. Dupont exploders were employed. The blasting charge at the quarry was calculated to average about ½ lb. of dynamite per cubic yard of rock. The charge was set off by means of electricity. The blasting gang on the day of observation consisted of one loader and two tampers.

The oil consumption of the drills was about 3 quarts each per shift. The power consumption was from 30 to 40 kw.-hrs. per drill per shift. There was one 8-hour shift per day.

Table I gives the time study of drills at the quarry.

From Table I it appears that the average cutting speed was 0.135 ft. per minute. The ratio of cutting time to total time was 0.511 and the ratio of idle time (including the time for shifting drills) to cycle time was 0.167.

TABLE III. TIME STUDY OF DRILLING PERFORMANCE OF ELECTRIC-AIR TRIPOD DRILLS AT THE DAM

Observations recorded in minutes and seconds

	No. of obs.	Minimum M. S.	Average M. S.	Maximum M. S.	Actual time M. S.	Consumed time per cent of total time
Drill cutting	12	6-25	11-35	21-24	139-00	58.9
Raising drill	11	0-10	0-51	2-00	9-22	4.0
Loosening chuck	12	0-00a	0-09	25	1 45	0.7
Removing bit	12	0-00b	-29	1-10	5-42	2.4
Balling hole	10	1-13	1-36	-42	16-01	6.8
Putting bit in hole	10	0-18	0-45	2-09	7-34	3.2
Inserting bit in chuck	10	0-0	0-2	0-55	4-08	1.7
Tightening chuck	11	0-02	0-14	0-40	2-30	1.1
Getting started	10	0-02	0-1	0-4	2-08	0.9c
Cycle totals		8-42	16-47	32-10	188-10	79.7
Shifting drill	2	4-02	12-00	15-00	28-48	12.24
Miscellaneous delays	8	0-17	2-24	9-40	19-12	8.1
Total					236-10	100.0

Linear feet drilled, 26.4 ft., average depth of holes, 11 ft., total working time, 3 hrs. 56 mins. 10 secs. The cutting speed was 0.190 ft. per minute, ratio of cutting time to total time was 0.589, ratio of idle time (including shifting of drills), to cycle time was 0.255. a Chuck loosened by one man simultaneously with raising of drill by other. b Bit removed by one man simultaneously with raising of drill by other. c The percentage of "Cycle Total" is higher in this case than at the quarry, due mainly to the fact that the delivery of sharpened steels to the drillers was more prompt. d For the same reasons as given in the note under "Shifting Drill" in the quarry time study, the time for shifting properly chargeable in this case is $\frac{26.4}{11} \times 12 = 28 - 48$.

Based on the above performance at the quarry, the costs given in Table II per linear foot drilled and per cubic yard loosened have been deduced. One drill cut 31 ft. in 447 mins. 53 secs., equivalent to 200 ft. by 6 drills in one day of 8 hours. The average spacing of the holes being 17.5 by 16 ft., the corresponding cubic yards loosened was

$$\frac{200 \times 17.5 \times 16}{27} = 2{,}070 \text{ cu. yd}$$

In Table II no account has been taken of contractor's overhead charges, superintendence, storage, repairs, preparatory costs, insurance, charity, accidents, legal or medical expenses, etc. The low cost per cubic yard is due to the unusually wide spacing of the holes, which were loaded with a heavy charge of dynamite.

Table III gives the time study of drills at the dam.

Based on the performance, at the dam pit, of one drill doing 26.4 lin. ft. in 236 mins. 10 secs. six drills would accomplish about 320 ft. per day.

TABLE II.—STANDARD BASIS OF COSTS AT THE QUARRY.

	Rate.	Amount.	Cost per lin. ft.	Cost per cu. yd
6 drillers	$2.50	$ 15.00		
6 driller helpers	1.75	10.50		
		$ 25.50	12.75	1.23
1½ blacksmith	3.00	4.50		
1½ blacksmith helpers	1.75	2.63		
2 nippers	1.50	3.00		
2 mules	1.50	3.00		
		$ 13.13	6.57	0.64
Total labor (drilling)		$ 38.63	19.32	1.87
Coal, 500 lbs.	3.50	0.87		
Oil, 3 qts. per drill	0.30	1.35		
Power, 35 kw-hr	0.01	2.10		
		$ 4.32	2.16	0.21
Total drilling cost		$ 42.95	21.48	2.08
Interest and depreciation, 2 per cent per month		7.70	3.85	0.37
		$ 50.65	25.33	2.45
3 powdermen	2.00	6.00		
1,035 lbs. dynamite	0.12	124.20		
25 exploders	0.03	0.75		
		$130.95	65.47	6.32
Total		$181.60	90.80	8.77

During observation, the spacing of holes at the dam was very irregular. The drills were at work on the cut-off trench on the upstream side of the dam. The wide cut, at a rough estimate, appeared to be nearly 200 ft. in width, but the cut-off trench was only about 15 or 16 ft. in width and averaged about 11 ft. deeper than the main cut. The spacing of holes was very close and the blasting charges were correspondingly light. The outer rows of holes had a spacing of only 6 ins., which resulted in practically smooth walls.

The total drilling cost at the quarry for six drills per day was $50.65, including interest and depreciation. On the same basis of costs the performance at the pit would be at the rate of $\frac{\$50.65}{320} = 15.8$ cts. per linear foot, as against 25.33 cts. at the quarry. Of course, for comparative purposes, the blasting costs should be omitted, inasmuch as the amount of powder per linear foot would be much lower at the dam, due to the closer spacing and lighter loading of the holes.

For similar reasons, and because of the irregular spacing at the pit, there would be little value in a comparison of costs per cubic yard excavated. The cost per linear foot drilled is less at the pit than at the quarry for the reasons that the cutting speed was greater and the ratio of cutting time to total time was greater.

A Comparative Cost Study of Steam and Electric Equipment for a 1,500-Ton Coal Mine.

The comparative power cost study given here contains figures of wider interest than the special case considered. They are from a report by Ralph D. Brown, chief engineer of the O'Gara Coal Co., Harrisburg, Ill. The report in abstract follows:

TABLE I.—COST OF PRESENT STEAM PLANT AT GREENRIDGE MINE, GREENRIDGE, ILL.

Item—	Cost installed.	Salvage value.	Rate of depreciation, per cent.	Interest, per cent.	Fixed charge per day.
One pair Litchfield hoisting engines, 20x30, 8-in. drum	$ 2,500.00	$ 750.00	5	5	$0.685
Four Erie boiler, 72 ins. by 18 ft., 125 hp	6,500.00	1,000.00	10	5	3.109
Two new boilers to be added in place of two above	3,200.00	......	..	5	.....
One Erie generator, 19x20, 200 hp	1,850.00	750.00	6	5	.559
One fan engine belt connected, 10x12	450.00	125.00	5	5	.128
One shaker screen engine, 10x20	350.00	100.00	5	5	.095
One shop engine, 8x12	250.00	75.00	5	5	.068
One A. C. generator, 250 volts, 600 amp., 150 kw	1,850.00	800.00	6	5	.559
Three boiler feed pumps: two 7x5x6, one 9x5¼x10	800.00	400.00	5	5	.218
One boiler house	850.00	50.00	6	5	.256
Four smoke stacks, 60 ft. high	700.00	......	20	5	.48
One reservoir	5,000.00	......	..	5	.685
Total	$21,600.00	$5,050.00			$6.84

The Greenridge mine of the Middle States Coal Corporation is locatel on the C. & A. and C. & N. W. Rys. at Greenridge, Ill., in the central coal field of the state. The present rated capacity is near 1,500 tons per 8-hour shift and this is a fair figure for future development. The seam being mined will average 6 ft. in thickness of a good grade of steam coal. The hoisting shaft is 341 ft. from the ground to top of coal and the total hoist to the dumping point is 378 ft.

TABLE II.—ESTIMATED COST OF ELECTRICAL EQUIPMENT FOR 1,500-TON MINE.

Item—	Cost installed.	Rate of depreciation, per cent.	Interest, per cent.	Fixed charge per day.
300 hp. hoist motor, 1 fly wheel generator set with drum control, switches and wiring complete	$14,000	5	5	$3.808
150 kw. motor generator set for haulage, etc., with switchboard and wiring complete	4,000	5	5	1.096
One fan motor with switches and auto. starter, 25 to 50 hp	950	6	5	.283
One 10 to 20 hp. screen motor with switches and starter complete	600	5	5	.161
One 5 hp. shop motor with wiring complete	100	5	5	.027
Two 10 hp. pump motors with electric pump and wiring	750	7	5	.244
Total	$20,400			$5.62

NOTE—After deducting 10 per cent on $5,509, the salvage value of the steam plant, we have the fixed charge on the electric installation, $4.32.

At present all coal produced is by hand, no machines being employed. Electric motor haulage has been installed and has proved a great saving. The top equipment and machinery is adequate, thoroughly modern and kept in good repair. The question of economy has inspired a thorough investigation as to the saving possible from a complete electrifying of the plant and also the substitution of machines for pick work. The plant at present is a going concern. The territory remaining to be exhausted at the present capacity will last 15 to 18 years, which is the life of the plant.

TABLE III.—COST OF STEAM POWER FOR GREENRIDGE MINE, WITH A PRODUCTION OF 1,500 TONS IN 8 HOURS.

Item—	Cost.
Three firemen, 8 hours each, at $2.62 per shift	$ 7.86
Two extra top men for boiler, coal and looking after pumps	4.48
16 tons of boiler coal at $1.05	16.80
Repairs to boilers and machinery	3.34
Repairs to electric machinery	1.00
Cylinder oil, waste, etc	1.25
Lubricating oil	.50
Repairs to buildings and conveyors	.50
Interest and depreciation (see Table I)	6.84
Total	$42.57

200 working days per year at a cost of $42.57 is a total cost of $8,514.00.

Whatever improvements are made at this time are a deferred charge against operation and their cost must be amortized at the end of 18 years. It is evident that a definite period remains in which the proposed installation must show a profit or be abandoned.

TABLE IV.—COST OF STEAM POWER AT GREENRIDGE MINE ON IDLE DAYS.

Item—	Cost.
1 fireman at 8 hours, at $2.62 per shift	$ 2.62
Ten tons of boiler coal at $1.05	10.50
Repairs to boilers and machinery	3.34
Repairs to electrical machinery	1.00
Cylinder oil and waste	0.25
Lubricating oil	0.25
Repairs to buildings and conveyors	0.50
Interest and depreciation (See Table No. 1.)	6.84
Total	$25.30

165 days at $25.30 gives an idle day cost for the year of $4,174.50.

TABLE V. ESTIMATED COST OF ELECTRIC POWER FOR GREENRIDGE MINE WHEN PRODUCING 1,500 TONS PER DAY.

Item or unit—	Hours' duty.	KWH.	Cost per day.	Cost per ton
50-KW. fan motor, 25 pct. load	24	300	$ 4.50	$0.003
150-KW. motor generator set for haulage, shop motor, pumps, lights and shaker screens, 65 pct. load for 8 hours, 20 pct. load for 2 hours	10	840	12.60	0.008
250-HP. hoist with motor generator set 900 hoists, 5,000 lbs., each in 8 hours; 100 hoists, 3,000 lbs., each in 2½ hours	10½	1,100	16.50	0.011
Repairs, including rewinding of armatures	..	...	1.75	0.001
Oil and miscellaneous supplies	..	...	1.25	0.009
Fixed rental for power at $100 per month	..	...	3.34	0.002
Fixed charges, interest and depreciation, on cost of electric machinery, minus the value of the salvage of the steam plant.	..	...	4.32	0.002
Fixed charges on steam plant to be removed, taking out the value of the pond and new boilers to be installed, with interest at 5 pct. and depreciation 2 pct	..	...	3.12	0.00
Total cost per day	..	...	$47.38	$0.031

Two hundred days at $47.38 per day gives the total cost of power for working days of $9,47

There are two separate propositions to be considered: First, a complete substitution of electric power for steam as a prime mover for all machinery, with operation continuing on the pick and hand mining basis. Second, a complete substitution of electricity for steam as in case number one and the additional substitution of electric machines for the present pick mining method.

In arriving at the values in Table I, actual costs have been taken wherever possible and the remaining figures taken from costs of similar plants. The plant as taken refers to producers of power and does not include such machinery as fans, shaker, screens, underground motors, and other items which would remain in operation after the change.

TABLE VI.—ESTIMATED COST OF ELECTRIC POWER FOR 1,500-TON MINE FOR IDLE DAYS.

Unit or item—	Hours' duty.	KWH.	Cost per day.
50-KW. fan motor, 25 pct. load	24	300	$ 4.50
150 KW. motor generator set for haulage, shop motor, pumps, lights and shaker screens, 25 pct. load, 2 hours' duty	2	75	1.13
250-HP. hoist motor with motor generator set, 30 hoists of 3,000 tons each	8	40	0.60
Electric repairs, including rewinding of armatures, etc	..	..	1.5
Oil and miscellaneous supplies	..	..	0.2
Fixed rental for power at $100 per month	..	..	3.34
Fixed charged interest and depreciation on cost of electric machinery minus the value of the salvage of the steam plant	..	..	4.3
Fixed charges, interest and depreciation on steam plant to be removed after taking out the value of the pond and new boilers to be installed, with interest at 5 pct. and depreciation 2 pct	..	..	3.1
Total cost per idle day	..	..	$18.7

165 days at $18.75 gives us an estimated cost of electric power on idle days of $3,093 per day

There has been considerable difficulty at this mine to obtain a constant supply of water suitable for evaporation. The cost of this item has been from $5 to $15 per day during dry periods of three months' duration. In order to assure a permanent supply a value of $5,000 has been placed on ponds of ample capacity which could be constructed on company land. This value includes the estimated value of the submerged lands as well as the labor cost of excavation and embankment. The average working day was chosen for the unit of comparison and all tables have been figured on this basis, unless otherwise stated. The electric power required to produce 1,500 tons in eight hours is simply an estimate based on the result of calculations and on tests made at the Jefferson plant of the Middle States Coal Corporation at Springfield. A few assumptions were necessary but are based on good practice. All power costs were then figured from the charge of 1½ ct. per K. W. H. and an additional fixed flat rate of $100 per month. Certain premiums and deductions from this price were to be allowed to be computed in the usual manner based on the maximum demand. We have considered however that a balance could be struck and that the fixe charge and a rate of 1½ ct. per K. W. H would be a fair value.

Taxes and insurance will be practically the same in either case and have not been in luded in any of the computed charges. Fixed charges were figured considering the life of he plant to be 15 to 18 years and that at the nd of that period the scrap value of the plant would simply pay for its removal and the settlement of minor charges incidental to the abandonment of the property.

Summing up the results shown in the various tables we find that the saving per year n favor of the electric installation amounts o only $122.75. This small profit would not warrant a change for purely economical reasons.

The economic value of the mining machine s beyond question in certain localities. Its value in this particular case can be figured only as a saving in power. Basing an estimate on previous experience we consider 400 K. W. H. a fair figure for the power consumed per day by the electric cutting machines required for this mine. The power cost at 2.2 cts. per K. W. H. (the estimated present cost) would amount to $8.80 per day. The power cost on 400 K. W. H. at the rate of 1.5 cts. per K. W. H. would be $6 per day. The total saving per year on power required for mining machines using a 200 working day unit would be $560. This is a very small item considering the capital invested.

CONCLUSION.

There are many small delays to a small steam plant which can hardly be figured exactly and yet have a certain economical value. For example, many claim that the loss of time due to frequent repairing of a small plant can be eliminated by purchasing power from a central station. This is no doubt true, but when we consider that one of many expensive armatures may become unfit for operation and also bear in mind the skill and time necessary to repair one of these units, the margin of difference does not appear so great. Again others claim that any amount of power is available when necessary, a condition impossible with small plants. This argument will not hold in the case of Greenride mine, for with the present method of mining all the power required is always available.

In conclusion it may be said that unless unforsen conditions should develop the present development of power by the use of steam can hardly be improved upon, economically, by means of the substitution of electricity from a central power station. Certain other benefits may accrue from such a change but they are not worthy of consideration.

Water Works

Specifications for Plant and Material to Sterilize the Buffalo Water Supply by Hypochlorite of Lime or Liquid Chlorine as an Alternative.

On March 18 bids will be received by the Department of Public Works of Buffalo, N. Y., for furnishing and installing a complete plant for the treatment of the city water supply with hypochlorite of lime, similarly for treatment with liquid chlorine and also for furnishing the hypochlorite or liquid chlorine necessary for the treatment of the water from date of installation of apparatus to Jan. 1, 1915. The use of hypochlorite of lime or chlorine gas are alternative propositions and the city will determine which it considers most advantageous for its use. While hypochlorite is now used in 600 American cities, liquid chlorine has not as yet been widely adopted. The applications of the latter form of chlorine have been so successful, however, and the use of the liquified gas possesses so many advantages that it is very likely many cities which are not at present sterilizing their water supplies, or that now have only temporary plants, will desire to receive alternate bids between hypochlorite of lime and liquid chlorine when installing new plants or when replacing present temporary with permanent plants. This view of the matter prompts us o give, at this time, the substance of the specifications under which the bids at Buffalo will be received. These specifications first describe the present works for collecting and conveying the water to the pumping station nd then state what the city will furnish, as well as what the bidder must furnish. Such other general information as a bidder would require are given. While it is unnecessary to oint out that conditions vary, the Buffalo pecifications are well drawn and will doubtless prove helpful in a general way to other water department officials who contemplate eceiving bids of the character here considered. Many of the facts given relative to the Buffalo supply works are interesting aside rom their special significance in connection with the installation of a sterilization plant.

DESCRIPTION OF INTAKE

The intake pier for the water of the city f Buffalo is situated in Lake Erie at the head f Niagara River. It is circular in form, 120 . outside diameter, the walls are 20 ft thick nd lined inside and out with [illegible] in. steel late. This leaves an interior chamber 70 ft 1 diameter into which the water enters hrough 12 ports, each 6x6 ft., controlled by gate at each port. In the center of this hamber leading to the tunnel is a shaft 13 t. in diameter constructed of [illegible] in. plate bove the bottom of the chamber, the water eing 22 ft. in depth. The water enters the haft through four gates. The shaft extends) ft. below the bottom of the chamber into ie tunnel. This part of the shaft is built in limestone rock and is lined with concrete. The tunnel is 12 ft. in diameter, about 6,600 ft. in length to the shaft on the main land, is built through limestone and is lined throughout with concrete. From this land shaft to the new pumping station (a distance of about 400 ft.) the water flows through canals, partly roofed and partly open, 6 ft. in width, 28 ft. in depth, the water being 21 ft. in depth. The canal is lined with concrete and is connected to the suction wells, which are also lined with concrete. From the shaft on the main land to the present pumping station is about 4,286 ft., the water flowing through a concrete-lined tunnel 9 ft. in diameter.

The intake pier has a concrete floor 20 ft. in width and about 200 ft. in length surrounding the well about 13 ft. above mean water level. There is another floor about 25 ft. above mean water level about the same dimensions as the lower floor. Over the tunnel shaft will be an iron platform octagonal in shape and about 16 ft. across.

There is now installed in the intake pier two boilers each of 100 HP. There is also a 10-KW. generator and engine for lighting, also cables extending through the tunnel giving telephone connection with the pumping station.

The water in Lake Erie is disturbed by windstorms, so that from 60 to 90 days each year it is more or less roily, the roil being composed of fine sand and clay. The remainder of the year it is reasonably clear.

The quantity of water used daily varies from 120 to 200 m.g.d. The plant to be installed must be of size to treat 200,000,000 gals. in 24 hours at a maximum rate of 1 part per 1,000,000 gals. of free or available chlorine, and to be readily adjusted to reduce such application to as low as 0.2 part per 1,000,000 gals.

BIDDING CONDITIONS.

The city will furnish and install any necessary steam pipes for heating, or power from boilers, to a convenient point for attachment to apparatus. The city will also furnish the following: Pumps for water supply, piping and storage tanks; electric power or wire at a convenient point; apparatus for removal and disposal of lime sludge from convenient discharge; and all necessary housing around apparatus.

PLANS AND DESCRIPTIONS.

Each proposal for apparatus will be accompanied by plans and a description of plant and all available appliances.

All applications of chlorine in solution must be made in the intake shaft at least 10 ft below the mean water level.

All parts and appliances to be in contact with hydrated chlorine must be made of such material or manufacture as not to be corroded by such solution. If mechanical parts cannot all meet this condition a statement must be made of such and the probable duration and cost of replacement, and the construction must be such as can readily be replaced by spare parts.

There are lead covered telephone and power cables passing through the tunnel and up the shaft and it is desired to make application of chlorine at such a point as to minimize danger to these cables.

All controlling appliances for controlling discharge between solution tank or gas container and point of application must be in duplicate to admit ready use of either set of appliances.

The control of the apparatus must be such as to allow the application of the predetermined amount of free or available chlorine at a certain rate per hour and must be sufficiently automatic to continue such rate of application for 8 hours without more than 10 per cent of variation from such a rate or such other allowable per cent or variation as the bidder may guarantee. All rates of application shall be capable of independent check by weight per 8-hour period, or less.

The rate of application will be controlled by telephonic advices from the pumping station as to rate of pumpage and under advice of the health department, based on analysis before and after treatment, but it is expected that there will be on the pier at all times a competent person to make minor examinations and supervise apparatus.

Owing to variety of apparatus the city desires full competition, but in general will demand that all material and appliances furnished hereunder shall be of the most suitable material and the best quality used for such purposes.

DISINFECTING MATERIALS.

All hypochlorite of lime must contain at least 35 per cent of available chlorine. Payment will be made in accordance with the test of the same, the price paid to be increased in proportion for any increase in percentage and reduced in double proportion, or entirely rejected for any decrease in percentage as the deputy water commissioner may direct. The hypochlorite must be shipped in suitable tight retainers, the bidder to state the style of container and the weight of the same.

Chlorine gas must be shipped in suitable containers and shall be a highly purified and especially prepared chlorine for use as a sterilizing agent. It must be anhydrous and free from all impurities. The bidder must state the capacity of the container in which it will be furnished. All containers must have arrangement for ready connection to the apparatus, and each container must be plainly and permanently numbered and marked with its tare weight.

OPERATION.

The contractor will control the apparatus of all equipments for a period of 30 days after installation and such operation must be to the satisfaction of a duly appointed representative of the city. All chlorite or chlorine gas which is required will be furnished by the city provided there is no careless waste in its use.

EXAMINATION OF WATER.

On Feb. 3, 1914, the water was in about its maximum turbidity and a test was made as follows:

	P. p. m.
Turbidity, silica scale	60
Total solids	214
Organic and volatile matter	49
Mineral solids	165

A microscopic examination of the sediment from this sample shows the presence of diatoms, a few algea, some decomposed vegetable tissues, a few hydras and considerable fine clay. A small amount of sand is also present.

For the information here given we are indebted to Mr. Henry L. Lyon, Deputy Water Commissioner.

Design and Construction of a Reinforced Concrete Fence for Reservoir Protection at Allegheny County Home, Woodville, Pa.

The Board of Directors of the Poor of Allegheny County, Pennsylvania, recently completed a concrete lined reservoir for the storage of water for the institution at Woodville. It was thought wise, considering the nature of the institution and the infirmities of the inmates, to construct a fence around this reservoir. Accordingly a reinforced concrete fence was designed for this purpose by Mr. J. G. Chalfant, county engineer, to whom we are indebted for the information here given. The fence is 551 ft. long and 8 ft. high. It consists of 34 panels of 16 ft. each and has one 6 ft. reinforced concrete gate. A view of a portion of the completed fence is shown in Fig. 1.

The posts are 12 ins. square and 13 ft. 6 ins. long, the lower 2½ ft. being reduced in section to 6 ins. square. They are reinforced with four ¾-in. rods placed one in each corner and extending from the top of the post to the bottom of the 6-in. x 6-in. section, and four ½-in. rods extending from the main body of the post into the 6-in. x 6-in. portion to insure against breakage in handling. The panel palings are 3½ ins. x 5 ins., reinforced with two ¼-in. rods. These rods are pointed on the upper end and are allowed to project above the top of the concrete. The posts and palings were built during the winter in a shop where heat was available. The mixture used was one part Universal Portland cement and two parts Ligonier stone screenings passing a ¼-in. screen. As soon as the concrete was sufficiently hard to allow the forms to be removed the surface was finished by rubbing with a block of wood. During the spring and summer of 1913 the units were hauled to the work as required.

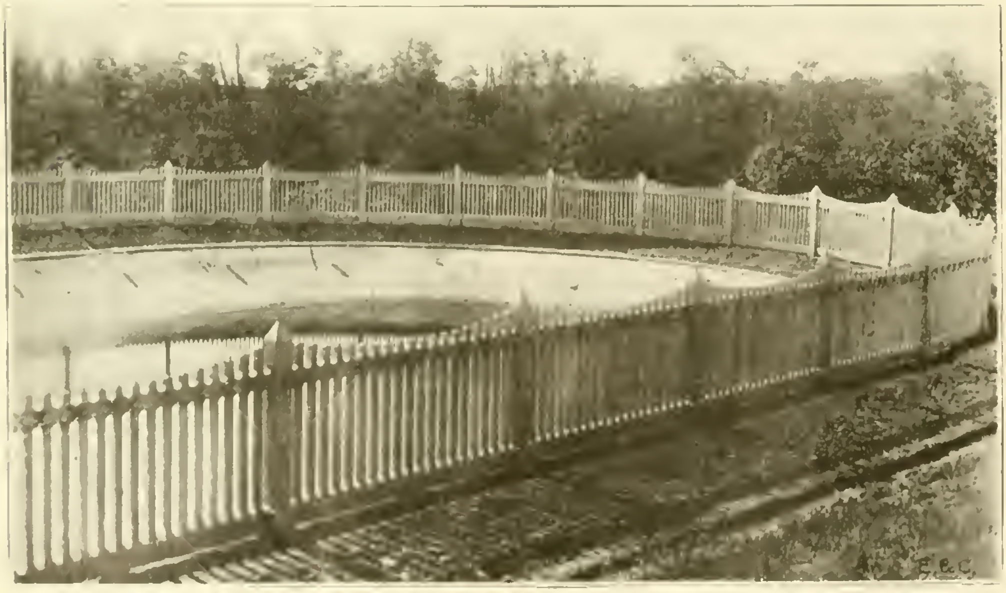


Fig. 1. View of Reinforced Concrete Fence for Water Supply Reservoir Protection at Allegheny County Home, Woodville, Pa.

The upper rail is 6 by 8 ins. in cross section, reinforced with two ¾-in. rods, while the lower rail is 8 by 12 ins. and has the same reinforcing. It is supported at the middle by a concrete post poured simultaneously with it. The rails rest in insets in the post, which are lined with tarred felt, thus providing for expansion and contraction and for any slight settlement that might occur. Each panel consists of 19 palings spaced 5⅝ ins. in the clear. The detail of a 16-ft. panel is shown in Fig. 2.

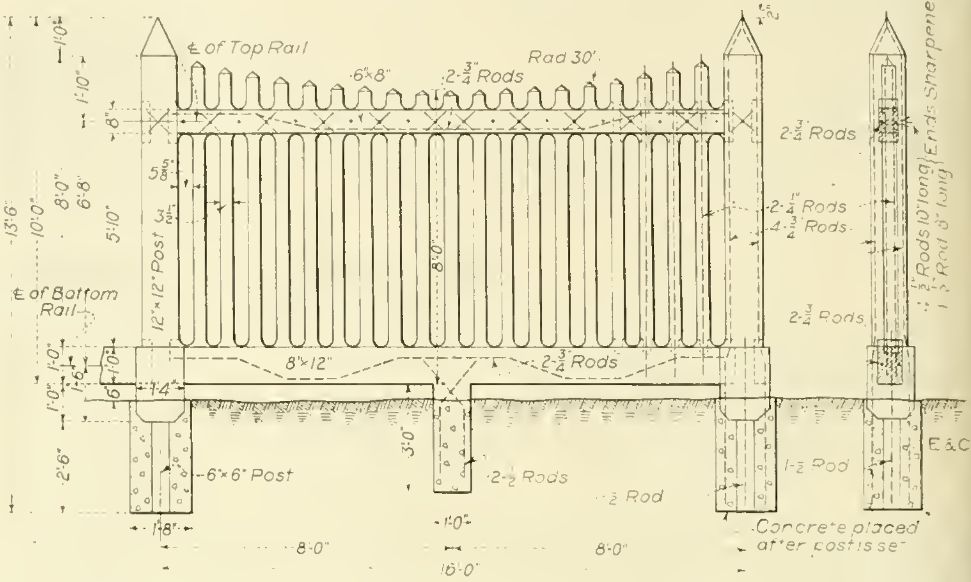


Fig. 2. Detail of 16 ft. Section of Reinforced Concrete Fence at Woodville, Pa.

In the construction of the fence the following method of procedure was found to be the most satisfactory. The work was carefully staked out and holes were dug for the posts and the intermediate supports. The posts were set to line and grade, their tops being 10 ft. above the ground level. Concrete was poured around the posts, encasing the 6-in. by 6-in. portion and securing the projecting ends of the ¾-in. reinforcing rods. Two 3 by 12-in. planks were clamped to the posts to support the palings and for a form for the upper rail. The palings were then placed, being supported by short 2x4-in. pieces clamped to them and resting on the plank. Two longitudinal 2x4's were then placed and on top of these were placed the half rounds that formed the lower fillet in the top rail. On these the rail forms were placed. The back form consisted of a single piece, while the front form was made of two pieces in order to allow for pointed iron rods forming clusters that project from the top rail. These clusters consist of four ½-in. bent rods 10 ins. long, and one ½-in. straight rod 8 ins. long, which add to the appearance of the fence and also to the difficulty that would be experienced in scaling it. The reinforcement was then placed and the whole made ready for concreting. Concrete was mixed in the proportion of one part Universal cement and two parts Ligonier stone screenings passing a ¼-in. screen. The rails were rubbed in the same manner as were the posts and palings. Upon completion of the fence the entire surface was rubbed with a carborundum brick and water. This finish gives a uniform surface without danger of subsequent hair cracking.

The gate, which is of reinforced concrete, was poured in a form under a shed and was swung into place with a derrick. It is of the same design as the fence, thoroughly reinforced with steel against sagging. The detail of the gate is shown in Fig. 3. Two 1-in. round rods secured in the gate post terminate in eyes. Two similar rods run through each rail of the gate and a bolt is dropped through the eyes upon which the gate swings. An iron wheel has been placed at the free end of the gate which rolls on a concrete slab, thereby relieving the post and gate of a portion of the strain resulting from the weight of the gate.

The cost of the fence is given below. The only skilled labor required was that of two first-class mechanics, a carpenter and a cement finisher, who were employed on the work. All common labor was performed by the inmates of the institution. The actual cost of this fence complete was $3.20 per linear foot.

An estimate based on the experience gained in this work, and assuming labor at $2 per day and material at the same price as at present, would indicate that assuming 500 ft. or more of the fence were built, that the cost would be about $4.07 per linear foot of fence, as follows:

Estimate of cost of 1 panel 16 ft. long, including 1 post.

Concrete materials	$ 7.20
Steel	7.56
Two men, 2 days at $5	20.00
Six men, 2 days at $2	24.00
	$58.76

	Per lin. ft
$\frac{\$58.76}{16}$ =	$3.67
All forms complete $\frac{\$200.00}{500}$ =	.40
Total estimated cost per foot	$4.07

The figures of actual cost given appear relatively high as compared with the estimated cost of the work if done entirely by paid labor. This is due to the inefficient labor of the inmates which caused the time of the two skilled laborers to become a much greater item than would have been the case had the fence been built entirely by paid labor under competent supervision.

CONSTRUCTION NOTES.

Forms.—Only one post form was required. Ten paling forms were used, making one-half of one set of palings at one time. These small paling forms could be used at least twice in one day, making with one post a complete section of fence 16 ft. long. All posts and palings were cast in a shop at some distance from the point of erection and the work was done during the winter. Upon assembling these pieces at the reservoir in the spring, the posts were first set to correct line and grade. All alternate 16-ft. panels were left open at first and built in after the forms for the completed panels were removed. Two complete sets of forms were required for the top and bottom rails and intermediate supports. This enabled one section of 16 ft. to be completed and another section to be set up and ready for concreting each day.

Protection Against Too Rapid Drying.—The most important precaution to take in work of this kind is for the proper protection of the fence from the direct rays of the sun, both at the time of placing the concrete and afterwards while the concrete is drying out. For this purpose a light cloth screen set above the posts to keep off the sun was used during concreting. As soon as possible after placing the concrete in the top and bottom rail forms, they were covered with wet gunny sacks and, after removal of forms on the following day these sacks were kept on and thoroughly wet with a hose several times a day. This was continued for a week. The only trouble experienced was that in a few instances proper care was not used in protecting new work from the sun. This resulted in hair cracks showing up which were afterwards with difficulty removed by rubbing. Too much emphasis cannot be laid on this feature in exposed concrete work of this kind.

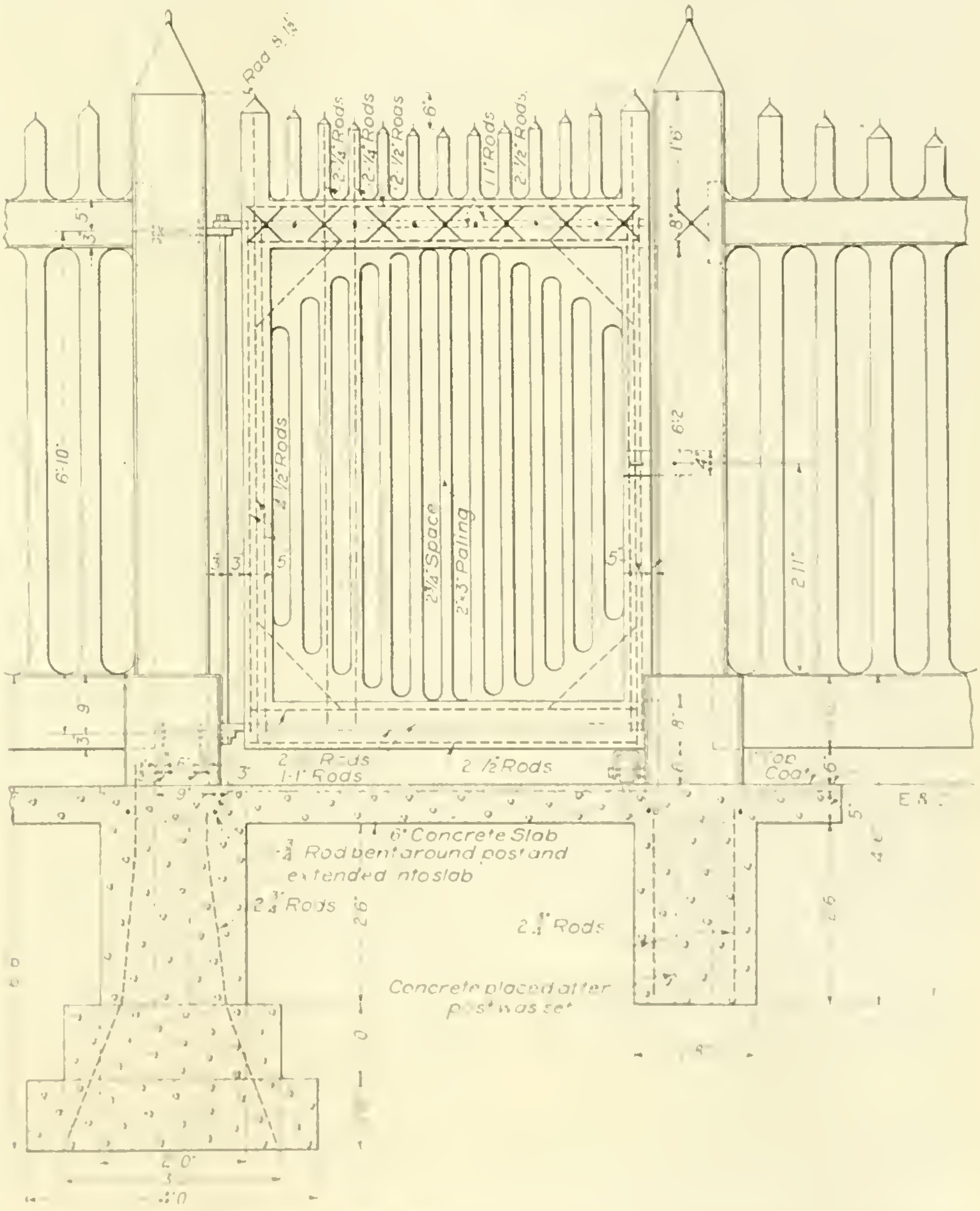


Fig. 3. Detail of Gate and Gateposts of Reinforced Concrete Fence at Woodville, Pa.

Appointment of an Engineer as Master in Chancery to Adjust Dispute Between City and Water Company at Little Rock, Ark.

The city of Little Rock brought suit in the Pulaski Chancery Court last October against the Arkansas Water Co., asking for the appointment of a receiver on account of the unsatisfactory condition of the water supply.

The court appointed Edward Flad, hydraulic engineer, St. Louis, Special Master in Chancery, to investigate and report, with power to administer oaths, to examine witnesses and to require the production of all documents, books and papers in the possession of either party bearing upon the issues.

With the consent of both parties the Master made a personal investigation and study of the situation. Surveys were made under his direction, and all available sources of supply were considered. The scope of his investigations and the nature of his findings are here indicated.

Shallow wells were considered as one of the possible sources of additional water supply. The water furnished by the shallow wells in the vicinity of Little Rock is generally fairly well suited for domestic and manufacturing purposes, but some of the wells deliver hard water containing noticeable quantities of iron in solution. A sufficient supply from shallow wells could be developed only by the expenditure of large sums of money, and, at best, there would always be considerable doubt as to the quality of the water and the permanency of the supply.

Deep wells were also given some consideration as a source of supply. The cost of such a supply could be determined only after a series of tests on wells to be sunk for that purpose. Such an investigation would have cost from $10,000 to $15,000. While it was considered probable that a sufficient supply could be developed from deep wells within a reasonable distance of the city, such a supply would always be more or less uncertain both as to permanency and quality. Since other available sources were free from these objections deep wells were not further considered.

In his investigation of sites for the construction of impounding reservoirs Mr. Flad examined six locations and estimated the cost of obtaining the additional water supply therefrom. Taking into consideration the contract between the city and the water company and the cost of developing an impounded supply it was considered unreasonable to require the Arkansas Water Co. to develop such an impounded supply except under an agreement providing for a readjustment of the terms of the franchise so that a reasonable return might be expected on the additional investment involved in excess of the expenditures required to develop a proper additional supply from the Arkansas River, the source now utilized.

Mr. Flad reported that a satisfactory supply of water can be obtained from the Arkansas River provided settling basins and filters of sufficient capacity are constructed and provided that a selective reservoir be constructed of a sufficient capacity to permit by-passing the river at times of excessive turbidity or chlorine. Certain additions to and alterations in the present plant of the Arkansas Water Co. will enable the company to provide a satisfactory supply of clear, pure and wholesome water free from objectionable quantities of chlorine. These changes in plant as recommended by Mr. Flad and as agreed upon by the parties to the suit are as follows: (a) A dam and selective reservoir to be constructed at a point about five miles west of the present pumping station. The capacity of this reservoir to be approximately 300,000,000 gals. (b) A pipe line of 11,000,000 gals. daily capacity from the reservoir to the pumping station. (c) An additional settling reservoir of a capacity of not less than 2,500,000 gals. (d) Additional filters of a capacity of not less than 4,000,000 gals. per day.

The date of the construction of the various additions to the present system, as above outlined, will depend upon the rate of increase of water consumption and upon the results attained by the water company in obtaining clear water free from excessive amounts of turbidity and chlorine. The findings further provide that the water company shall have the option of constructing at this time a selective reservoir at the new site with a capacity of 150,000,000 gals. instead of 300,000,000 gals. upon condition that the dam be so constructed as to readily permit of enlargement to increase the storage capacity to 300,000,000 gals. The reservoir construction must begin within three months and, together with the pipe line construction, must be completed by the middle of 1916. Half of the additional filters must be in service by 1919 and the other half four years later. The new settling basin must be completed by 1927. These dates are contingent upon the furnishing of pure and wholesome water in the meantime, otherwise the proposed works must be constructed when such water can no longer be furnished without them. Not over 400 p. p. m. of chlorine will be allowed. Mr. Flad will pass upon all plans for the works here outlined. The new works will cost about half a million dollars. The cost of the report here under consideration was shared equally by the city and the water company.

The "selective" reservoir will probably be the first of the kind to be constructed in this country. This reservoir, as stated, will be formed by placing a dam across a ravine and will be filled with rain water from a catchment area of about 2½ squares miles. Provision will also be made to pump river water into the reservoir, if required, at times when the river water is of satisfactory quality.

At certain periods of the year the Arkansas River contains noticeable and objectionable quantities of salt and at other periods its turbidity is such as to prevent its ready clarification. Both of these objectionable features are introduced by a tributary, the flow of which is, at times, a considerable part of the flow of the river. The object of the selective reservoir is to furnish an available supply of pure water during these periods of excessive salt or suspended matter, the supply ordinarily being taken from the Arkansas River, which, excepting during the periods referred to, furnishes a very satisfactory water after being settled and filtered.

On Feb. 11 the tentative findings of the Master, as above outlined, were submitted at an informal meeting, with the statement that if the tentative findings were not acceptable to both parties the Master would proceed to hold formal sessions, and examine witnesses under oath, the final findings to be based on the evidence so submitted.

Both parties to the case accepted the tentative findings, and the Court, being notified to that effect, issued a decree in accordance therewith on Feb. 13.

The appointment of an engineer, with technical training and experience, as Master in this case, resulted in a speedy settlement acceptable to both parties and with a minimum of costs. The action of the Court in appointing an engineer as Special Master with power to investigate and report, was somewhat unusual; the general practice in such cases being to appoint a lawyer with little or no knowledge of the technical questions involved.

The results in this case would seem to justify the action of the Court in departing from established custom in this respect.

The Use of Hypochlorite of Lime for Disinfecting Public Water Supplies Now Firmly Established as Good Practice.

The adoption of hypochlorite of lime for disinfecting public water supplies has spread very rapidly since its introduction at the Bubbly Creek filter plant in Chicago in 1908. There must be, at this time, fully 600 cities in this country which use hypochlorite for the purpose here under consideration. The use and misuse of this germicide and some of the statistical evidences of its successful application in various cities were discussed by Mr. C. A. Jennings, superintendent of filtration at the Union Stock Yards, Chicago, in a recent paper before the Illinois Water Supply Association, from which the information here given is taken

Wherever a proper survey was made and it was decided by capable authorities that hypochlorite treatment was needed and the chemical was properly applied thereafter, good results have followed. The writer knows of no city following this procedure that has found it necessary to discontinue the use of this treatment. It is true that in some cities hypochlorite treatment has come into bad repute with the water consumers because of a taste in the treated water at times. In the experience of the writer, such cases are always due to either a poorly designed plant, improper application, expecting a removal of bacteria beyond what was practical, or to the fact that hypochlorite treatment was not adapted to remedy the particular water troubles.

Hypochlorite has its sphere of usefulness and beyond that it is practically useless. It is to be used primarily to eliminate from a water supply the disease producing organisms of intestinal origin, such as typhoid fever, paratyphoid, dysentery, etc. The efficiency of the treatment will vary inversely with the turbidity of the water. Hypochlorite is now used in conjunction with rapid sand filters, slow sand filters, coagulation and sedimentation, plain sedimentation, impounding reservoirs, well water and alone.

With the layman of today so widely awakened to matters pertaining to health, more and more thought is being given by him to the purity of his drinking water. He is beginning to realize that the physical properties of a water do not necessarily denote the health-giving properties.

It is acknowledged that hypochlorite treatment is not a substitute for filtration, but there are some water supplies that do not need filtration because of their physical condition. Within the past two months Dr. Rudolph Hering of New York, reporting on the Buffalo water supply, said: "It is my opinion that sterilization by either method (hypochlorite of lime or liquid chlorine gas) will safeguard the public health at a very much lower cost than the adoption of filtration at this time. As soon as the city is prepared to pay the cost of giving the water at all times an excellent appearance, filtration is the system to be adopted." It is often a question of whether the results accomplished by filtration of a given water supply are worth the expenditure, provided, of course, that the water as furnished the consumers is free from pathogenic organisms. Many lake cities are situated similarly to Buffalo, as regards water supply. Erie, Pa., and Evanston, Ill., are now completing the installation of rapid sand filters. Cleveland, O., has broken ground for the first of two filtration plants. Filtration has been recommended for the water supply of Chicago. All of these cities have been using hypochlorite treatment for two years or more, producing a water that is bacteriologically safe and each city has reduced its typhoid fever death rate very appreciably.

Hypochlorite treatment is now firmly established because it has accomplished what its conservative advocates originally claimed for it. Its selective action for the germs of water-borne diseases has been proven by bacteriological analyses over a period of almost six years and by typhoid fever statistics during the same period. The writer has collected typhoid fever death rates from a number of cities and selected a few as being representative. The water supplies used include lake water with only hypochlorite, very turbid river water with coagulation, sedimentation and disinfection, impounding reservoirs with hypochlorite, plain sedimentation and hypochlorite and rapid sand filtration with disinfection. The statistics cover a maximum period of eleven years previous to disinfection and five years following its introduction. Obviously the latter figure is limited because hypochlorite was not introduced until 1908. Typhoid death rates for several cities are given in Table I, and Table II shows the comparative death rates from this disease before and after the introduction of hypochlorite.

TABLE I.—TYPHOID FEVER DEATH RATES IN EIGHT AMERICAN CITIES.

Name of City.	Aver. 1900-05.	1906.	1907.	1908.	1909.	1910.	1911.	1912.	1913.
Baltimore, Md.	36	34	41	31	23	42	26	22	23.7
Cleveland, Ohio	51	20	19	13	12	19	14	6	14
Des Moines, Iowa	*18.4	16.7	31.3	19.5	18	32.6	19	9.7	11.6
Erie, Pa.	28	47	78	61	29	39	197.0	8.7	18.3
Evanston, Ill.	..	..	21	32	28	23	27	16	13
Jersey City, N. J.	19	21.4	14.8	10	9	11.6	8	7.8	10.3
Kansas City, Mo.	48	38	40	35	23	43	24	12	23.8
Omaha, Nebr.	20	28	24	22	31	89	17	12	6.4

*The figures 18.4 for Des Moines is the rate for 1905 and not the average for the period 1900-05.

TABLE II.—AVERAGE TYPHOID FEVER DEATH RATES BEFORE AND AFTER INTRODUCTION OF HYPOCHLORITE TREATMENT.

	Began hypo.	Before hypo.		After hypo.	
Baltimore, Md.	June, 1911	1900-1910	35.2	1912-13	22.8
Cleveland, Ohio	Sept., 1911	1900-1910	35.5	1912-13	10
Des Moines, Iowa	Dec., 1910	1905-1910	22.7	1911-13	13.4
Erie, Pa.	Mar., 1911	1900-1910	38.4	1912-13	13.5
Evanston, Ill.	Dec., 1911	1907-1911	26	1912-13	14.5
Jersey City, N. J.	Sept., 1908	1900-1907	18.7	1909-13	9.3
Kansas City, Mo.	Jan., 1911	1900-1910	42.5	1911-13	20
Omaha, Nebr.	May, 1910	1900-1909	22.5	1911-13	11.8

Table II shows that for the 11 years previous to the use of hypochlorite, Baltimore had a typhoid fever death rate of 35.2. Eliminating the year 1911, during the last six months of which hypochlorite was used, the death rate was reduced to an average of 22.8 for the years 1912 and 1913 while using hypochlorite, a reduction of 35 per cent. Baltimore uses chemical coagulation and sedimentation and hypochlorite. A rapid sand filter plant is being installed at this time.

The average typhoid death rate in Cleveland, O., for the 11 years previous to the introduction of hypochlorite in September, 1911, was 35.5 as shown. The average for 1912 and 1913 while using hypochlorite, was 10, a reduction of 73 per cent. Cleveland uses no other treatment than disinfection at this time.

At Des Moines, Ia., the water supply is collected in filter galleries under the old bed of the river. Hypochlorite treatment was begun in December, 1910, during a typhoid epidemic. Table II shows that the average typhoid death rate for the years 1905-1910, inclusive, was 22.7 and for the three years while using this process it was 13.4 per 100,000, a reduction of 41 per cent.

During the winter of 1910-1911, Erie, Pa., experienced a typhoid epidemic of a very severe form. The death rate from this disease was 197 per 100,000 for the year 1911. Hypochlorite treatment was begun in March, 1911. Table II shows the effect that this treatment has had in reducing typhoid fever. The average typhoid death rate for the 11 years, 1900-1910, inclusive, was 38.4 and for 1912 and 1913, while treating the water supply, it averaged 13.5, a reduction of 65 per cent.

Evanston, Ill., is another city on the Great Lakes that suffered a typhoid fever epidemic in 1911, but it was prevented from assuming large proportions by the introduction of hypochlorite in December, 1911. An average typhoid death rate of 26 per 100,000 for the years 1907-1911, inclusive, dropped to 14.5 as the average for 1912 and 1913, a reduction of 41 per cent.

Jersey City, N. J., was the first city to adopt hypochlorite treatment for a municipal water supply, which was in September, 1908. The water is stored in an impounding reservoir, and then treated with hypochlorite. The average typhoid death rate for the years 1900-1907, inclusive, was 18.7 per 100,000. The average death rate since the introduction of hypochlorite for the years 1909-1913, inclusive, has been 9.34, which is a reduction of 50 per cent.

Kansas City had a typhoid death rate of 43 per 100,000 during 1910. Hypochlorite treatment was begun in January, 1911. The average typhoid death rate for the years 1909-1910, inclusive, was 12.5 per 100,000. During 1911, 1912 and 1913 the average was only 20.

This was a reduction of 53 per cent over the former rate.

The water supply of Omaha, Nebr., has been coagulated and settled in basins and then treated with hypochlorite since May, 1910, when disinfection was installed because of the large amount of typhoid fever in the city at that time. The typhoid death rate in 1910 was 89 per 100,000. The average typhoid fever death rate for 1900-1909, inclusive, was 22.5. For the years 1911, 1912 and 1913 the death rate averaged 11.8, a reduction of 47 per cent.

Summarizing, it is seen that in each city substantial reduction in typhoid fever was effected following the introduction of hypochlorite. These reductions have been consistent. For the eight cities, the maximum reduction was 72 per cent (Cleveland), and the minimum was 35 per cent (Baltimore). The average percentage reduction was 51, which is very satisfactory. The total population of the eight cities is 2,033,000. With an average reduction of 15.8 deaths per 100,000 population for each year and with a value of $5,000 for each life saved, the saving to these cities amounts to $1,606,000 per year. Certainly the hypochlorite treatment has proved a blessing.

Formerly it was thought that the water supply of any city with a typhoid fever death rate of 20 or less per 100,000 was above suspicion. This view has changed recently. Of the eight cities named only one had a typhoid death rate less than 20 previous to the use of hypochlorite (Jersey City with 18.7). The average was 30.2. The average since disinfection was begun is 14.4 and only one city (Baltimore, with 22.8) has rate above 20 per 100,000.

It is to be noted in conclusion that of the eight cities, six showed a higher typhoid death rate in 1913 than in 1912. It has been said that 1913 was a typhoid year. During 1910 and 1911 there were many typhoid epidemics. What the death rates in these cities would have been if hypochlorite had not been in use will never be known. However, it is safe to say that hundreds of lives were saved during 1913 that would otherwise have been sacrificed to that preventable disease, typhoid fever, had the water supplies not been disinfected with hypochlorite.

Discussion of Preventable Destruction of Child Life, in Winter and Spring, by Polluted Water Supplies.

In presenting arguments for safe water supplies it is customary to point out the disgraceful rates of typhoid fever in cities with polluted water supplies and to lay stress upon the almost invariably marked and prompt reduction in these rates following the substitution of a safe for a polluted supply. These arguments are unanswerable and usually to a thinking community effective. We must not conclude, however, that the reduction in typhoid fever is the only result desired and achieved. There is, in many instances, a marked reduction in the general death rate and especially in the rates for diarrhœa and enteritis of children. At the recent annual meeting of the Indiana Sanitary and Water Supply Association, Allan J. McLaughlin, Surgeon U. S. Public Health Service and director of field work of the International Joint Commission, presented instances and data bearing upon the relation of polluted water supplies to infant mortality. The following quotation is from Dr. McLaughlin's paper.

What factors enter into the composition of the group classified officially as diarrhœa and enteritis under two years? We do not know. Probably there are several entities involved. Typhoid fever, various types of dysentery, heat exhaustion and inanition. In spite of all the mystery which still enshrouds the etiology of summer diarrhœa of infants (whether its origin be climatic, dietetic or infectious), one fact stands out and is admitted by all, it should be an insignificant factor in our death rates for the winter and spring months. Its maximum prevalence should be in July and August and even in towns with an appalling yearly rate for "enteritis"; if the water supply is safe there should be little or no mortality from this cause in the cold months.

There is undeniable evidence of the part played by heat in infant mortality. This infant mortality in summer, although its etiology is still a matter of discussion, is a clearly defined entity which might be called "legitimate" enteritis of infants and seems to be in most cases entirely independent of the water supply. There is another type of enteritis of children to which I desire to call attention, which may occur at any time, which is associated with and probably dependent upon a sewage polluted water supply and which shows a very high rate of prevalence in the winter and spring months.

The public water supply of Wyandotte, Mich., is taken from a grossly polluted river. Not only are the enteritis rates in Wyandotte very high, but there is an abnormal prevalence in the spring months. For example, there is much more fatal "enteritis" in Wyandotte in March and April than there is in Benton Harbor in July and August. Benton Harbor has a public water supply from deep wells generally conceded to be safe.

St. Joseph, Mich., has a public water supply which is polluted at times. St. Joseph and Benton Harbor are sister cities on opposite banks of the St. Joseph River, at its mouth. The only essential difference in sanitary conditions exists in water supplies. The "enteritis" rate in St. Joseph for the winter and spring months is abnormal compared with Benton Harbor.

There is a very irregular curve for "enteritis" in Ludington, Mich. Two abnormal peaks in February and May suggest the agency of a polluted water supply. In 1913 Ludington treated the water supply with hypochlorite and there was not a single death from enteritis under two years of age during the year.

Many more examples in Michigan cities might be cited—for instance, Alpena, Marquette, Muskegon, Bay City and Port Huron, and I have no doubt a careful analysis of the seasonal prevalence of enteritis in Hammond, Whiting or Michigan City, Ind., for a period of years will show the same phenomena.

An interesting comparison of the seasonal prevalence of enteritis in Marquette and Sault Ste. Marie strengthens the opinion that a polluted water supply bears a definite relation to diarrhœa and enteritis under two years. Both cities had water supplies which were polluted at times. The pollution of the Marquette supply frequently occurred in the winter and spring months. The pollution of the Sault Ste. Marie supply is due entirely to boat traffic, hence is in evidence only from May to November. The rates for enteritis correspond with these periods, and there is an abnormal prevalence in Marquette in the winter and spring months, while the very high rates in Sault Ste. Marie are in evidence only in the summer and autumn.

It is not contended that a polluted water supply is a large factor in the summer diarrhœa of infants, but that in certain cities a polluted water supply has been responsible for many deaths in summer and for almost the entire mortality from enteritis during the winter and spring months. I will take one instance from New York State to show that this phenomenon is not confined to Michigan. The city of Cohoes took its public water supply from the grossly polluted Hudson River. Until July, 1911, this water was not purified. At that time a gravity mechanical filter was installed. The very high rate in Cohoes for enteritis under two years had been notorious for years. The average rate per 100,000 for the three years, 1909-1911, was 200. In 1912 this rate dropped to 80. The seasonal prevalence of enteritis in Cohoes had the characteristics of cities having polluted water supplies, viz., very high rates in the winter and spring months. How this winter and spring diarrhœa and enteritis of children was reduced is shown by the following tabulation:

ENTERITIS UNDER TWO YEARS AT COHOES, N. Y., BEFORE AND AFTER ADOPTING WATER FILTRATION.

Year.	Rate per 100,000.
1909	144
1910	192
1911	192
1912	40
1913	48

CONCLUSIONS.

1. In cities with polluted water supplies the rate for diarrhœa and enteritis under two years is abnormally high in the winter and spring months.

2. The substitution of a safe for a polluted public water supply results in a marked reduction of deaths of infants from diarrhœa and enteritis, especially during the months from December to June.

3. If any argument other than the record of water-borne typhoid were necessary to secure safe water supplies, the preventable destruction of child life in winter and spring months due to polluted water would seem to furnish that argument.

A Mistake in Repairing a Pressure Filter and Its Lesson.

To the Editor: Your idea that it would be of great benefit if engineers would publish their mistakes is excellent, but I am afraid impracticable. An engineer naturally likes to forget embarrassments of this kind and is always in hope that others will do likewise, but unfortunately for him the human memory is prone to retain the mishaps of others. There is nothing better for the education of the engineer than to be humiliated in his early days by the discovery of a more or less disastrous error.

The writer had one of these experiences in 1890 which beautifully illustrates the point in consideration, particularly as the error was one in elementary mechanics that should have been inexcusable.

A certain water company bought in Pittsburgh a second-hand National pressure filter, which they installed in the pumping station of a Georgia town. As soon as it was put under normal pressure it showed unmistakable signs of failure, and the writer was called in by an officer of the company to suggest a remedy. For this purpose he wrote to the local superintendent for complate data as to the construction of the filter. The information obtained showed that it was 10 ft. in diameter, 7 ft. high with flat top and bottom held together with 1-in. stay bolts. In the bottom was a foot or so of concrete, containing the effluent manifold system, and projecting from the top of this concrete were the little rose perforated strainers, each filled with quartz gravel.

A simple calculation based on the number, cross-section and strength of the bolts showed that the filter was designed for a working pressure of about 50 lbs. per square inch. But the pumping station hydrostatic pressure was about 125 lbs. The writer therefore recommended increasing the number of bolts, as the shell thicknesses were ample to withstand the working pressure.

The same officer of the company objected to the writer's recommendations on account of the cost and difficulty of breaking through the concrete and otherwise providing fittings for the bolts, and suggested reinforcing the filters from the outside by a system of beams and requested the writer to make such a design. After showing the disadvantages of this plan due to appearance and probable greater cost than the other plan, the design was made briefly as follows. To be on the safe side the beams were designed of sufficient strength to carry the maximum working pressure, without placing any dependence whatever on the 50 1-in. bolts already in place, and allowing for a maximum fiber stress of 15,000 lbs. per square inch in the steel I-beams. It required a flooring of 5-in. 10-lb. beams top and bottom and resting upon these on the top were two pairs of 15-in. beams and on the bottom two pairs of 12-in. beams. These were held together by four heavy bolts from top to bottom, using plate washers, all on the outside of the filters.

The work was properly installed by the local superintendent and the pressure was turned on gradually, when suddenly one of the 15-in. beams broke through in the center.

On receiving the letter from the superintendent giving the circumstances of the failure, the writer went over his calculations carefully and found they were all correct, but it was some time before he could divine the true cause of the break

In order for the beams to deflect in the center a sufficient amount to stress the outer fibers to their working load, it was necessary for the inside stay rods to elongate a sufficient amount to strain them beyond their elastic limit. The consequence was some of them broke, which caused a shock sufficient to crack the beam.

The remedy should have been to first cut through all of the interior stay bolts.

Fortunately, the loss was not great, but the lesson taught was as impressive as if the failure had been of many times the importance.

Very truly yours,

"Engineer."

Philadelphia, Pa., Feb. 20, 1914.

(Aside from the points brought out by our correspondent in the foregoing communication we desire to call attention to another point relative to the description of their mistakes and failures by designers. The opinion has been expressed, and is doubtless widely held, that an engineer endangers his reputation for care and accuracy when he describes one of his mistakes for publication. In order to bring out the fact that it is not essential that the name of the contributor be published we have in this case given our correspondent the pen name "Engineer," although he was willing that we publish his name. His identity is, of course, known to the editors and we will say that he is now chief engineer of an important engineering and construction company. We feel that the value to young engineers of the lesson brought out in this letter is in no wise diminished by the suppression of the correspondent's name.—Editors.)

Drainage and Irrigation

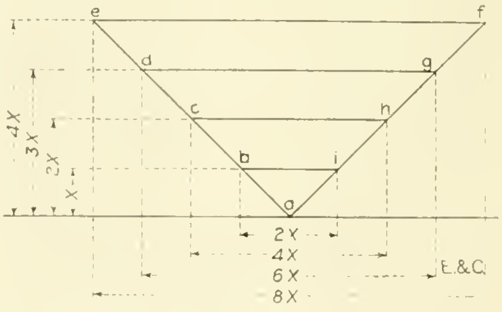
Drainage Ditch and Levee Tables for Level Section With Explanation of Method of Computation.

Contributed by E. S. Blaine, Cape Girardeau, Mo.

When we commenced estimating quantities for the ditches to be excavated for the Little River Drainage District, we encountered a condition not uncommon to drainage work, but not generally found in other kinds of work; namely, a country as nearly a plane as is possible for ground surface to be, with gradual slope of 1 ft. to the mile in a southwesterly direction. Even for the ditches having a maximum base width of 123 ft., the levels show so little difference in elevation in the width of the ditch for a large per cent of the length of the ditch, that this portion may be considered level. This made it practicable to use tables for level sections for nearly all of the ditches and levees, and to use tables calculated to tenths of a foot, not that it is practicable to dig a ditch or build a levee with a dipper dredge or a drag line excavator, within two or three tenths of a foot of grade, but because the contractors expect to receive pay for the given prism for a calculated depth as shown by the grade line on the profile and no less, while the district expects to pay for this calculated quantity and no more.

We were unable to get hold of tables calculated to tenths of feet, and as there were four levee sections with different crowns and slopes and some 45 different size ditches with base widths between 4 ft. and 123 ft., making tables for these looked like a pretty fair sized job, enough so, in our opinion, to attempt to find the easiest and most rapid method of calculating them. Mr. Wm. Mullholland's idea of using second differences and calculating them by addition clear through within the limits of the table, with a check on every fifth foot, appeared to be the most rapid and also the most accurate method and this method was used for the work. This second difference was obtained as follows:

Supposing the ditch section to have slopes of 1:1 and these slopes produced downward to an apex, thus giving a triangular section as shown in the figure below:



	1st Diff.	2nd Diff.
Area Triangle a b i=X^2		
	$3X^2$	
Area Triangle a c l=$2X\times2X=4X^2$		$2X^2$
	$5X^2$	
Area Triangle a d g=$3X\times3X=9X^2$		$2X^2$
	$7X^2$	
Area Triangle a e f=$4X\times4X=16X^2$		

Letting $X = .1$;

Then $2X^2 = .02$.

Volume of second difference for prism 100 ft. long and slopes 1:1 as above $= \frac{.02\times100}{27} =$ .074074+.

For proof with actual example, take a table for a section having 20 ft. base, slopes 1:1, starting with 5 ft. cut. Calculating prism for depth of 5 ft. and length of 100 ft. gives us:

5.0	Quantity.	First diff.	Second diff.
5.0	462.962963		
		11.148148	
5.0	474.111111		.074074
		11.222222	
5.2	485.333333		

As will be seen by the above solution, this second difference not only remains constant for prisms having the same slope, but also the second differences for other slopes are a multiple of this quantity .074074 if the slopes are the same on both sides of the section. If not the same on both sides, then it is a multiple of the average of the two slopes. The meaning intended to be conveyed here can be made more plain by an example, thus: For a section having 2:1 slopes the second difference is twice 0.0741 or 0.1482, for slopes of 3:1 it is 0.2222, and for section having 2:1 slope on one side and ½:1 on the other it is $\frac{½ + 2}{2}$ x 0.074074 or 0.0925925, or say 0.0926, which proved close enough for checking and correcting at 5 ft. intervals.

It will be observed that after adding this second difference to itself a number of times, which varies for different slopes, the number to the right of the decimal point repeats itself. This simplifies the work a great deal as will be shown. Taking the 1:1 slope and adding the second difference successively and discarding the integral numbers,

.0741
.1482
.2222
.2963
.3704
.4444
.5185
.5925
.6667
.7407
.8148
.8889
.9629
.0370
.1111
.1852
.2593
.3333
.4074
.4815
.5555
.6296
.7037
.7778
.8518
.9259
.0000

Using the table for base width of 20 ft. slopes 1:1, which was used as an example, the difference between yardage for 5 ft. cut and cut of 5.1 was 11.1481. The decimal portion of this difference is the second number in the table above. It will be seen then by using the numbers in the above table in their order and adding one to the integral portion of the difference when the fractions show

TABLE I.—VOLUME OF EXCAVATION IN CUBIC YARDS PER 100 LIN. FT. OF DITCH IN LEVEL SECTION; SIDE STOPES 1 ON 1; BOTTOM WIDTHS, 4 TO 123 FT.

Cut.	0	.1	.2	.3	.4	.5	.6	.7	.8	.9
BASE 4 FT.										
3.........	...	...	...	...	...	...	...	...	110	114
4	119	122	128	132	137	142	146	151	156	161
5.........	167	172	177	183	188	193	199	205	211	216
6.........	222	228	234	240	247	253	259	265	272	279
7	285	292	299	306	312	319	327	334	341	348
8	356	363	370	378	386	394	401	409	417	425
9.........	433	442	450	458	466	475	484	492	501	510
10.........	519	527	536	546	555	564	573	583	592	601
11.........	611	621	631	640	650	660	670	680	691	701
BASE 5 FT.										
2.........	52	55	59	62	66	69	73	77	81	85
3.........	89	93	97	101	106	110	115	119	124	129
4.........	133	138	143	148	153	158	164	169	174	180
5.........	185	191	196	202	208	214	220	226	232	238
6.	244	251	257	264	270	277	284	290	297	304
7.........	311	318	325	333	340	347	355	362	370	377
8	385	393	401	409	417	425	433	441	450	458
9.........	467	475	484	492	501	510	519	528	537	546
10......	556	565	574	584	593	603	612	622	632	642
11......	652	662	672	682	692	703	713	724	734	745
BASE 6 FT.										
2	59	63	67	71	75	79	83	87	91	96
3.........	100	104	109	114	118	123	128	133	138	143
4	148	153	159	164	169	175	181	186	192	198
5.........	204	210	216	222	228	234	241	246	253	260
6......	267	273	280	287	294	301	308	315	322	330
7......	337	344	352	360	367	375	383	391	399	407
8......	415	423	431	440	448	456	465	474	482	491
9......	500	509	518	527	536	545	555	564	573	583
10......	593	602	612	622	632	642	652	662	672	682
11......	693	703	713	724	735	745	756	767	778	789
12..	800	811	822	834	845	856	868	880	891	903
13.........	915	927	939	951	963	975	987	1,000	1,012	1,024
14	1,037	.	...	...	...	...	...	...	...	...
BASE 7 FT.										
9......	533	543	552	561	571	581	590	600	610	620
10	630	640	650	660	670	681	691	701	712	723
11.	733	744	755	766	777	788	799	810	822	833
12	844	856	868	879	891	903	915	927	939	951
13	963		.	..	...	...	...	...	...	...
BASE 8 FT.										
4	178	184	190	196	202	208	215	221	228	234
5	241	247	254	261	268	275	282	289	296	304

t:o be necessary, the work is put in such lpe that it may be done on a Burrough's Ading Machine, taking sub-totals for each lition. This makes it almost a mechanical ration. Example: Using above table for e of 20 ft., slopes 1:1, yardage for 5 ft. s found to be 462.9629

5.0 = 462.9629
11.1482
5.1 = 474.1111
11.2222
5.2 = 485.3333
11.2963
5.3 = 496.6296
11.3704
5.4 = 508.0000

5.4 = 508.0000
11.4444
5.5 = 519.4444
11.5185
5 6 = 530.9629
11.5925
5.7 = 542.5554
11.6667
5.8 = 554.2221
11.7407
5.9 = 565.9628
11.8148
6.0 = 577.7776

omputing yardage for cut of 6 ft. as check finds it to be 577.7778.

n the actual work we only used three imal places, finding this sufficiently acrate for intervals of 5 ft. and then corting for next 5 ft.

or tables of prisms having 2:1 slopes, operation was the same except that the cessive adding of the second difference this slope makes the decimal fractions ne in the following order:

.0000
.1482
.2963
.4444
.5925
.7407
.8889
0370
1852
3333

.4815
.6296
.7778
.9259
.0741
.2222
.3704
.5185
.6667
.8148
.9629
.1111

.2593
.4074
.5555
.7037
.8518
.0000

or table for prisms having 3:1 slope, the ference has only nine different decimal lings for any one width crown, so the table decimal endings previously given had to divided into three tables to be used inpendently of each other and the nine endgs repeated until the table had been comted.

slopes when wn is even ultiple of 3.	3:1 slope when crown is multiple of 3+1.	3:1 slope when crown is multiple of 3 1
.0000	.0370	.0741
.2222	.2593	.2963
.4444	.4815	.5185
.6667	.7037	.7407
8889	.9259	.9629
1111	1481	.1852
3333	.3704	.4074
.5555	.5925	.6296
.7778	.8148	8518

We also calculated a set of tables for a sm having slope of 2:1 on one side and :1 on other. This required a set of 54 ferent decimal fractions.

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ost of Excavating Drainage Ditch With a Drag Line Excavator.

ntributed by Ray S. Owen, Civil Engineer, Madison, Wis.

The following is a statement of the cost of cavating drainage ditches in Rock County, is. The machine used was a drag line edge with steam power, running on a track d by hand and propelled by pulling on ad man with the hoisting drum The p ating crew consisted of 1 runner, 1 fireın, 2 trackmen and 1 teamster.

The ditches are in a hay meadow, the soil ing about 2 ft. of muck underlaid by sand te ditches averaged 5 ft. deep with 1½ to 1 ope, the main ditch, 2.60 miles in length, ving a 6-ft. bottom with 21-ft top and e lateral 1.36 miles in length, having a 4-ft ttom with 19-ft. top. The total excavation mputed for the ditch was 53,019 cu yds ie soil caved very badly and a large amount excess material had to be excavated to get e specified prism clear of dirt The amount dirt actually moved was about 75,000 cu s.

The two ditches are not connected but pty into Sugar River at points about onearter mile apart. This arrangement nessitated a tear down and move of about

TABLE I—Continued

Cut.	0	.1	.2	.3	.4	.5	.6	.7	.8	.9
6	311	319	326	335	342	350	358	366	374	382
7	390	398	406	415	422	431	439	448	456	465
8	474	483	492	501	510	519	529	538	548	557
9	567	576	586	596	606	616	626	636	646	656
10	667	677	688	698	709	719	730	741	752	763
BASE 9 FT.										
5	269	266	273	281	288	295	303	310	318	326
6	333	341	349	357	365	373	381	390	398	406
7	415	423	432	441	449	458	467	476	485	494
8	504	513	522	532	541	551	561	570	580	590
9	600	610	620	630	641	651	661	672	682	693
BASE 10 FT.										
5	278	285	293	300	308	316	324	331	339	347
6	356	364	372	380	389	397	406	414	423	432
7	441	450	459	468	477	486	495	505	514	624
8	532	543	553	563	572	582	592	603	613	623
9	633	644	654	665	675	686	697	708	719	730
10	741	752	763	774	786	797	809	820	832	844
11	856	...	...	...	...	...	...	...	...	...
BASE 12 FT.										
2	104	110	116	122	128	134	141	147	153	160
3	167	173	180	187	194	201	208	215	222	230
4	237	244	252	250	267	275	283	291	299	307
5	315	323	331	340	348	356	365	374	382	391
6	400	409	418	427	436	445	455	464	473	483
7	493	502	512	522	532	542	553	562	572	582
8	593	603	613	624	635	645	656	667	678	689
9	700	711	722	734	745	756	768	780	791	803
10	815	827	839	851	863	875	887	900	912	924
11	937	950	962	975	988	1,001	1,014	1,027	1,040	1,053
12	1,067	1,080	1,093	1,107	1,121	1,134	1,148	1,162	1,176	1,190
13	1,204	1,218	1,232	1,246	1,261	1,275	1,289	1,304	1,319	1,333
14	1,348	.....	.....	.....	.....	.....	.....	.....	...	....
BASE 14 FT.										
4	267	275	283	291	300	308	317	326	334	343
5	352	361	370	379	388	397	407	416	425	435
6	444	454	464	474	484	494	504	514	524	534
7	544	555	565	576	587	597	608	619	630	641
8	632	663	674	686	697	708	720	731	743	755
9	767	779	791	803	815	827	839	851	864	876
10	889	902	914	927	940	953	966	979	992	1,005
11	1,019	1,032	1,045	1,059	1,072	1,086	1,100	1,114	1,128	1,142
12	1,156	1,170	1,184	1,198	1,212	1,227	1,241	1,256	1,271	1,285
13	1,300	.....	.....	.....	.....	.....	.....	....	....	....
BASE 20 FT.										
5	463	474	485	497	508	619	531	543	554	566
6	578	590	602	614	626	638	650	663	675	687
7	700	713	725	738	751	764	777	790	803	816
8	830	843	856	870	884	897	911	925	939	953
9	967	981	995	1,009	1,024	1,038	1,052	1,067	1,082	1,096
10	1,111	1,126	1,141	1,156	1,171	1,186	1,201	1,217	1,232	1,247
11	1,263	1,279	1,294	1,310	1,326	1,342	1,358	1,374	1,390	1,406
12	1,422	1,439	1,455	1,471	1,488	1,505	1,521	1,538	1,555	1,572
13	1,589	1,606	1,623	1,640	1,658	1,675	1,692	1,710	1,728	1,745
14	1,763	1,781	1,799	1,817	1,835	1,853	1,871	1,889	1,908	1,926
15	1,944	1,963	1,982	2,000	.....	.....	.....	.....	...	..
BASE 22 FT.										
8	889	903	917	931	946	960	975	989	1,004	1,019
9	1,033	1,048	1,063	1,078	1,093	1,108	1,124	1,139	1,154	1,170
10	1,185	1,201	1,216	1,232	1,248	1,264	1,280	1,296	1,312	1,328
11	1,344	1,361	1,377	1,394	1,410	1,427	1,444	1,460	1,477	1,494
12	1,511	1,528	1,545	1,563	1,580	1,597	1,615	1,632	1,650	1,667
13	1,685	1,703	1,721	1,739	1,757	1,775	1,793	1,811	1,830	1,848
14	1,867	.....	.....	.....	.....	.....	.....	....	..	..
BASE 24 FT.										
5	537	550	562	575	588	601	614	627	640	653
6	667	680	693	707	721	734	748	762	776	790
7	804	818	832	846	861	875	889	904	919	933
8	948	963	978	993	1,008	1,023	1,038	1,054	1,069	1,084
9	1,100	1,116	1,131	1,146	1,163	1,179	1,195	1,211	1,227	1,243
10	1,250	1,276	1,292	1,308	1,325	1,342	1,358	1,375	1,392	1,409
11	1,426	1,443	1,460	1,477	1,495	1,512	1,529	1,547	1,565	1,583
12	1,600	1,618	1,636	1,654	1,672	1,690	1,708	1,726	1,744	1,763
13	1,781	1,800	1,819	1,837	1,856	1,875	1,894	1,913	1,932	1,951
14	1,970	1,990	2,009	2,029	2,048	2,068	2,088	2,107	2,127	2,147
15	2,167	2,187	2,207	2,227	2,248	2,268	2,288	2,309	2,329	2,350
16	2,371	2,391	.....	.....	.....	.....	....	.	.	..
BASE 25 FT.										
5	556	569	582	595	608	621	635	648	662	675
6	689	703	716	730	744	758	772	787	801	815
7	830	844	859	873	888	903	918	933	948	963
8	978	993	1,008	1,024	1,039	1,055	1,070	1,086	1,102	1,117
9	1,133	1,149	1,165	1,181	1,198	1,214	1,230	1,247	1,263	1,280
10	1,296	1,313	1,330	1,347	1,364	1,381	1,398	1,415	1,432	1,449
11	1,467	1,484	1,502	1,519	1,537	1,555	1,572	1,590	1,608	1,626
12	1,644	1,663	1,681	1,699	1,718	1,736	1,755	1,773	1,792	1,811
13	1,830	1,849	1,868	1,887	1,906	1,925	1,944	1,964	1,983	2,003
14	2,022	2,042	2,062	2,081	2,101	2,121	2,141	2,161	2,182	2,202
15	2,222	2,243	2,263	2,284	.....	....	.			
BASE 27 FT.										
5	593	606	620	634	648	662	676	690	705	719
6	733	748	762	777	792	806	821	836	851	866
7	881	897	912	927	943	958	974	990	1,005	1,021
8	1,037	1,053	1,069	1,085	1,101	1,118	1,134	1,150	1,167	1,183
9	1,200	1,217	1,233	1,250	1,267	1,284	1,301	1,318	1,336	1,353
10	1,370	1,388	1,405	1,423	1,441	1,458	1,476	1,494	1,512	1,530
11	1,548	1,566	1,585	1,603	1,621	1,640	1,658	1,677	1,696	1,714
12	1,733	1,752	1,771	1,790	1,809	1,829	1,848	1,867	1,887	1,906
13	1,926	1,946	1,965	1,985	2,005	2,025	2,045	2,065	2,085	2,106
14	2,126	2,146	2,167	2,187	2,208	2,229	2,249	2,270	2,291	2,312
15	2,333	2,354	2,376	2,397	2,418					
BASE 30 FT.										
4	504	518	532	546	561	[illegible]	589	604	619	633
5	648	663	678	693	708	723	738	754	769	784
6	800	816	831	847	863	879	895	911	927	943
7	959	976	992	1,008	1,025	1,041	1,058	1,075	1,092	1,109
8	1,126	1,143	1,160	1,177	1,194	1,212	1,229	1,247	1,265	1,283
9	1,300	1,318	1,336	1,354	1,372	1,390	1,408	1,426	1,445	1,463
10	1,481	1,500	1,519	1,537	1,556	1,575	1,594	1,613	1,632	1,651
11	1,670	1,690	1,709	1,728	1,748	1,768	1,787	1,807	1,827	1,847
12	1,867	1,887	1,907	1,927	1,947	1,968	1,988	2,008	2,029	2,050
13	2,070	2,091	2,112	2,133	[illegible]	2,175	2,196	2,217	2,239	2,260
14	2,281	2,303	2,324	2,346	[illegible]	2,390	2,412	2,434	2,456	2,478
15	2,500									
BASE 32 FT.										
4	533	548	563	578	[illegible]	608	624	639	654	670
5	685	701	716	732	748	764	780	796	812	828
6	844	861	877	894	910	927	944	960	977	994
7	1,011	1,028	1,045	1,062	1,080	1,097	1,115	1,132	1,150	1,167
8	1,185	1,203	1,221	1,239	1,257	1,275	1,293	1,311	1,330	1,348
9	1,367	1,385	1,404	1,423	1,442	1,461	1,479	1,498	1,517	1,536
10	1,556	1,575	1,594	1,611	1,633	1,653	1,672	1,692	1,712	1,732
11	1,752	1,772	1,792	1,812	[illegible]	[illegible]	1,873	1,891	1,914	1,935

TABLE 1.—Continued.

Cut.	.0	.1	.2	.3	.4	.5	.6	.7	.8	.9
12	1,956	1,976	1,997	2,018	2,039	2,060	2,081	2,103	2,124	2,145
13	2,167	2,188	2,210	2,231	2,253	2,275	2,297	2,319	2,341	2,363
14	2,385	2,407	2,430	2,452	2,475	2,497	2,520	2,543	2,565	2,588
15	2,611	.....	.....	.....	.....	.....	.....	.....	.....	.....
BASE 38 FT.										
5	796	814	832	850	868	886	904	923	941	959
6	978	996	1,015	1,034	1,052	1,071	1,090	1,109	1,128	1,147
7	1,167	1,186	1,205	1,225	1,244	1,264	1,284	1,303	1,323	1,343
8	1,363	1,383	1,403	1,423	1,444	1,464	1,484	1,505	1,525	1,546
9	1,567	1,587	1,608	1,629	1,650	1,671	1,692	1,714	1,735	1,756
10	1,778	1,799	1,821	1,843	1,864	1,886	1,908	1,930	1,952	1,974
11	1,996	2,019	2,041	2,063	2,086	2,108	2,131	2,154	2,176	2,199
12	2,222	2,245	2,268	2,291	2,315	2,338	2,361	2,385	2,408	2,432
13	2,456	2,479	2,503	2,527	2,551	2,575	2,599	2,623	2,648	2,672
14	2,696	2,721	2,745	2,770	2,795	2,819	2,844	2,869	2,894	2,919
15	2,944	2,970	.....	.....	.....	.....	.....	.....	.....	.....
BASE 40 FT.										
5	833	852	871	889	908	927	946	965	984	1,003
6	1,022	1,042	1,061	1,080	1,100	1,119	1,139	1,159	1,179	1,199
7	1,219	1,239	1,250	1,279	1,299	1,310	1,340	1,360	1,381	1,402
8	1,422	1,443	1,464	1,485	1,506	1,527	1,548	1,569	1,591	1,612
9	1,633	1,655	1,676	1,698	1,720	1,742	1,764	1,786	1,808	1,830
10	1,852	1,874	1,896	1,919	1,941	1,964	1,987	2,009	2,032	2,055
11	2,078	2,101	2,124	2,147	2,170	2,194	2,217	2,240	2,264	2,287
12	2,311	2,335	2,359	2,383	2,407	2,431	2,455	2,479	2,503	2,537
13	2,552	2,576	2,501	2,626	2,650	2,675	2,700	2,725	2,750	2,775
14	2,800	2,825	2,851	2,876	2,901	2,927	2,952	2,978	3,004	3,030
15	3,056	3,082	3,108	3,134	3,160	3,186	3,212	3,239	3,265	3,292
16	3,319	3,345	3,372	3,399	3,426	3,453	3,480	3,507	3,534	3,562
17	3,589	3,616	3,644	3,671	3,699	3,727	3,755	3,783	3,811	3,839
18	3,867	3,895	3,923	3,951	3,980	4,008	4,037	4,066	4,094	4,123
19	4,152	4,181	4,210	4,239	4,268	4,297	4,327	4,356	4,385	4,415
20	4,444	4,474	4,504	.....	.....	.....	.....	.....	.....	.....
BASE 50 FT.										
4	800	822	843	865	886	908	930	952	974	996
5	1,019	1,041	1,063	1,085	1,108	1,130	1,153	1,176	1,198	1,221
6	1,244	1,267	1,291	1,314	1,337	1,360	1,384	1,407	1,431	1,454
7	1,478	1,502	1,525	1,549	1,573	1,597	1,621	1,646	1,670	1,694
8	1,719	1,743	1,768	1,792	1,817	1,842	1,867	1,891	1,916	1,942
9	1,967	1,992	2,017	2,043	2,068	2,094	2,119	2,145	2,171	2,196
10	2,222	2,248	2,274	2,300	2,327	2,353	2,379	2,406	2,432	2,459
11	2,485	2,512	2,539	2,566	2,592	2,619	2,647	2,674	2,701	2,728
12	2,756	2,783	2,811	2,838	2,866	2,894	2,921	2,949	2,977	3,005
13	3,033	3,062	3,090	3,118	3,147	3,175	3,204	3,232	3,261	3,290
14	3,319	3,347	3,376	3,406	3,435	3,464	3,493	3,523	3,552	3,582
16	3,611	3,641	3,671	3,700	3,730	3,760	3,790	3,820	3,851	3,881
16	3,911	3,942	3,972	4,003	4,033	4,064	4,095	4,126	4,156	4,187
17	4,219	4,250	4,281	4,312	4,344	4,375	4,407	4,438	4,470	4,502
18	4,533	4,565	4,597	4,629	4,661	4,693	4,726	4,758	4,791	4,824
19	4,856	.....	.....	.....	.....	.....	.....	.....	.....	.....
BASE 60 FT.										
7	1,737	1,764	1,792	1,820	1,847	1,875	1,903	1,931	1,959	1,987
8	2,015	2,043	2,071	2,100	2,128	2,156	2,185	2,214	2,242	2,271
9	2,300	2,329	2,358	2,387	2,416	2,445	2,475	2,504	2,533	2,563
10	2,593	2,622	2,652	2,682	2,712	2,742	2,772	2,802	2,832	2,862
11	2,893	2,923	2,953	2,984	3,015	3,045	3,076	3,107	3,138	3,169
12	3,200	3,231	3,262	3,294	3,325	.....	.....	.....	.....	.....
BASE 70 FT.										
9	2,633	2,666	2,699	2,731	2,764	2,797	2,830	2,863	2,896	2,930
10	2,963	2,996	3,030	3,063	3,097	3,131	3,164	3,198	3,232	3,266
11	3,300	3,334	3,368	3,403	3,437	3,471	3,506	3,540	3,575	3,610
12	3,644	3,679	3,714	3,749	3,784	3,819	3,855	3,890	3,925	3,961
13	3,996	4,032	4,068	4,103	4,139	4,175	4,211	4,247	4,283	4,319
14	4,356	4,392	4,428	4,465	4,501	4,538	4,575	4,611	4,648	4,685
15	4,722	4,759	4,796	4,834	4,871	4,908	4,946	4,983	5,021	5,059
16	5,096	5,134	5,172	5,210	5,248	5,286	5,324	5,363	5,401	5,439
17	6,478	5,516	.....	.....	.....	.....	.....	.....	.....	.....
BASE 73 FT.										
6	1,756	1,787	1,819	1,850	1,882	1,914	1,946	1,978	2,010	2,042
7	2,074	2,106	2,139	2,171	2,204	2,236	2,269	2,301	2,334	2,367
8	2,400	2,433	2,466	2,499	2,532	2,566	2,599	2,633	2,666	2,700
9	2,733	2,767	2,801	2,835	2,869	2,903	2,937	2,971	3,005	3,040
10	3,074	3,109	3,143	3,178	3,212	3,247	3,282	3,317	3,352	3,387
11	3,422	3,457	3,493	3,528	3,564	3,599	3,635	3,670	3,706	3,742
12	3,778	3,814	3,850	3,886	3,922	3,958	3,995	4,031	4,068	4,104
13	4,141	4,177	4,214	4,251	4,288	4,325	4,362	4,399	4,436	4,474
14	4,511	4,549	4,586	4,624	4,661	4,699	4,737	4,775	4,813	4,851
15	4,889	4,927	4,965	5,004	5,042	5,081	5,119	5,158	5,196	5,235
16	5,274	5,313	5,352	5,391	5,430	5,469	5,509	5,548	6,588	5,627
17	5,667	5,706	5,746	5,786	5,826	5,866	5,906	5,946	5,986	6,026
18	6,067	6,107	6,148	6,188	6,229	6,269	6,310	6,351	6,392	6,433
19	6,474	6,515	6,556	6,598	6,639	6,681	.....	.....	.....	.....
BASE 74 FT.										
10	3,111	3,146	3,181	3,216	3,251	3,286	3,321	3,357	3,392	3,427
11	3,462	3,499	3,534	3,570	3,606	3,642	3,678	3,714	3,750	3,786
12	3,822	3,859	3,895	3,931	3,968	4,005	4,041	4,078	4,115	4,152
13	4,189	4,226	4,263	4,300	4,338	4,375	4,412	4,450	4,488	4,525
14	4,563	4,601	4,639	4,677	4,715	4,753	4,791	4,829	4,868	4,906
15	4,944	4,983	5,022	5,060	5,099	5,138	5,177	5,216	5,255	5,294
16	5,333	5,373	5,412	5,451	5,491	5,531	5,570	5,610	5,650	5,690
17	5,730	5,770	5,810	5,850	5,890	5,931	5,971	6,011	6,052	6,093
18	6,133	.....	.....	.....	.....	.....	.....	.....	.....	.....
BASE 75 FT.										
5	1,481	1,513	1,545	1,576	1,608	1,640	1,672	1,704	1,736	1,768
6	1,800	1,832	1,865	1,897	1,929	1,962	1,995	2,027	2,060	2,093
7	2,126	2,159	2,192	2,225	2,258	2,292	2,325	2,358	2,392	2,426
8	2,459	2,493	2,527	2,561	2,595	2,629	2,663	2,697	2,731	2,766
9	2,800	2,834	2,869	2,904	2,938	2,973	3,008	3,043	3,078	3,113
10	3,148	3,183	3,219	3,254	3,289	3,325	3,361	3,396	3,432	3,468
11	3,504	3,540	3,576	3,612	3,648	3,684	3,721	3,757	3,793	3,830
12	3,867	3,903	3,940	3,977	4,014	4,051	4,088	4,125	4,162	4,200
13	4,237	4,274	4,312	4,350	4,387	4,425	4,463	4,501	4,539	4,577
14	4,615	.....	.....	.....	.....	.....	.....	.....	.....	.....
BASE 80 FT.										
5	1,574	1,607	1,641	1,674	1,708	1,742	1,775	1,809	1,843	1,877
6	1,911	1,945	1,979	2,014	2,048	2,082	2,117	2,151	2,186	2,221
7	2,256	2,290	2,325	2,360	2,395	2,431	2,466	2,501	2,536	2,572
8	2,607	2,643	2,679	2,714	2,750	2,786	2,822	2,858	2,894	2,930
9	2,967	3,003	3,039	3,076	3,112	3,149	3,186	3,223	3,259	3,296
10	3,333	3,370	3,108	3,445	3,482	3,519	3,557	3,594	3,632	3,670
11	3,707	3,745	3,783	3,821	3,859	3,897	3,935	3,974	4,012	4,050
12	4,089	4,127	4,166	4,205	4,244	4,282	4,321	4,360	4,399	4,439
13	4,478	4,517	4,556	4,596	4,635	4,675	4,715	4,751	4,794	4,834
14	4,874	4,914	4,954	4,994	5,035	5,075	5,115	5,156	5,196	5,237
15	5,278	5,319	5,359	5,400	5,441	5,482	5,524	5,565	5,606	5,647
16	[illegible],689	5,73[illegible]	5,772	5,814	5,855	5,897	5,939	5,981	6,023	6,065
17	6,107	6,150	6,192	6,234	6,277	6,319	6,362	6,405	5,448	6,490
18	6,533	6,576	6,619	6,663	6,706	6,749	.....	.....	.....	.....
BASE 81 FT.										
8	2,637	2,673	2,709	2,745	2,781	2,818	2,854	2,890	2,927	2,963
9	3,000	3,037	3,073	3,110	3,147	3,184	3,221	3,258	3,296	3,333

three miles from the end of one ditch, after it was completed, to the other ditch, and a set up.

The prices given include freight on machine from Madison, Wis., to Sterling, Ill., the operation, moving, repair, etc., of the machine during the work, and the tearing down and delivery of the machine on board cars at Sterling, which is about 8 miles from the job. The rent of 2 cts. a yard included the furnishing, by the owner of the dredge, of sheaves and cable, which was a large item as the sand wore them out very rapidly. The cost of coal, teaming and moving is rather large, because of very bad roads when the outfit was moved out in the spring and the deep sand through which the coal was hauled during the summer. The unit prices are given for the contract yardage and for the actual yardage.

Items—	Cost per cu. yd. Contract yardage.	Actual yardage.
Rent of dredge	$0.0200	$0.0141
Labor	0.0535	0.0376
Coal	0.0090	0.0063
Express and freight	0.0028	0.0020
Bond and liability insurance	0.0022	0.0015
Livery and carfare	0.0019	0.0013
Oil	0.0005	0.0003
Teaming and moving	0.0110	0.0080
Tools, supplies, repairs, lumber	0.0027	0.0020
Miscellaneous	0.0011	0.0009
Total cost	$0.1047	$0.0740

Linings for Small Storage Reservoirs.

Contributed by C. R. Sessions, San Francisco, Cal.

CLAY LINING.

Probably the cheapest and best lining for a small reservoir is a clay puddle. A fairly good clay can nearly always be found within reasonable hauling distance. A small amount of clean sand or gravel will not affect the good quality of the puddle, but loam, roots, leaf mold and in fact all vegetable matter must be religiously excluded. Adobe is unfit for a lining of this kind because it bakes out when dry and deep shrinkage cracks form, which allow the water to run through carrying away the lining in the neighborhood of the crack before the whole surface can re-form and be tight again. In time adobe in the presence of water will be carried out as mud through the outlet of the reservoir.

The clay is hauled or scraped in and evenly distributed over the surface of the reservoir to a thickness of 3 or 4 ins. or more, depending upon the pressure or depth of water in the reservoir and the quality of the clay used for puddle. Puddling can be accomplished by merely filling carefully with water or by rolling or tramping the surface with horses or sheep after thoroughly sprinkling. The largest item in the cost of such a lining is the cost of the clay and this depends of course upon the thickness that must be used and the length of the haul. One cent per square foot will cover the cost for ordinary conditions of say a haul of 1½ to 1 miles and a thickness of 3 to 4 ins. When the reservoir is exposed to the action of the wind and waves are formed which will tend to cut away the lining at the water line, the slope should be covered with a strip of rock or rip-rap. This protection will cost about ⅝ ct. per square foot in addition to the cost of the lining itself. Of course this rock surface extends only around the slope at the water line. Road oil or crude oil may be sprayed upon the outer slope or surface of the levee to prevent the growth of vegetation and the activity of squirrels, gophers, etc. The use of oil as a lining will be discussed later and under the head of asphaltic linings.

PLASTERED COBBLE LINING.

A lining is sometimes made by setting up large broken rock, cobbles or river boulders in a mortar and covering the surface with a thin layer of cement plaster of say ½ in. in thickness. One part of lime to five parts of clean sand makes a good mortar and hydrated lime added to the cement plaster in the proportion of 10 per cent of the volume of the cement used will fill the smaller voids not filled by the cement and tend to make the mixture denser and therefore more nearly waterproof. If the reservoir is anything but very small one, trouble will be experienced from shrinkage cracks caused by the setting of the plaster so that this character of lining is no better than the usual cement lining where a surface similar to sidewalk is laid over the bottom and slopes of the reservoir. The only advantage of the cobble lining over the common cement lining comes from the fact that the sides or slopes of the reservoir may be made steeper (say ½ to 1) and deeper and therefore a smaller reservoir is required. Sometimes also the river boulders or cobbles are right at hand or may have been gathered up in clearing the land as was true in one case that the author has in mind. Here the soil was sandy and the cobbles came in very handy in holding the slope. A plastered cobble lining can be figured to run about 13 cts. per square foot.

CONCRETE LINING.

For a concrete lining the surface of the reservoir should be thoroughly settled by sprinkling and rolling. If the reservoir is a small one shrinkage may be taken care of by wire mesh reinforcement and the lining put in one piece. For larger reservoirs the concrete should be run in in slabs of about 200 sq. ft. separated by expansion joints. These joints are made by laying wooden strips, with beveled edges so that they can be easily removed, between the slabs and filling the space with asphaltum when the wooden pieces are taken out. The asphaltum should have as high a melting point as is consistent with proper ductility and adhesion to the concrete surface. This is accomplished generally by mixing with the asphaltum the proper amount of limestone dust and sand. The mixture has the ultimate melting point of the binder—the asphaltum—but the sand and dust give a sort of mechanical stiffness which keeps the joint filling from flowing until it has been thoroughly heated for some time. Expansion joints should have a substantial backing of broken rock so that the water pressure will not force out the asphaltum. When the reservoir is built in against a hill or sloping ground a broken rock or open tile drain should be provided to carry off the seepage water that may accumulate behind the lining and cause an upward pressure when the reservoir is empty. The proper amount of cement, sand or gravel will depend upon the material to be had in the vicinity. If good, clean material can be got, one part of cement to six or seven parts of gravel or sand and rock will give a concrete of ample strength and dense enough to be fairly water tight. A concrete that is dense is not only the most impervious but the strongest so that the addition of hydrated lime in the proportion of 10 per cent of the volume of the cement used will fill the smaller voids not filled by the cement and make the concrete more nearly water tight. The best proportion for the materials to be used in any locality have been determined and can generally be gotten from the firms supplying the materials. If the particular sand and gravel have not been used or the material firm is not reliable a mechanical analysis can be made to determine the best and most economical proportions. Nine cents per square foot in place will about cover a 3½-in. concrete lining.

When the cost of pumping is high and the cost of the water will warrant refinement, it may pay to plaster the above cement lining with a plaster coat of a cement mortar composed of one part of cement to two parts of sand and applied to the surface of the concrete as soon as possible after the [illegible] is in place and worked well with a trowel to secure a good bond with the concrete and make a water tight surface. A plaster about ½ in. thick will cost about 4 cts. per square foot. A thicker plaster would probably be no better and might check badly. Concrete linings as thin as 2 ins. have been used on irrigation ditches satisfactorily at a cost of about 6 cts. per square foot, but this is pretty thin work and is easily broken in the case of settlement and is almost impossible to repair. When a reservoir can be fenced and there is no danger of stock getting into it and breaking the

TABLE I. Continued.

Cut.	.0	.1	.2	.3	.4	.5	.6	.7	.8	.9
10	3,370	3,407	3,445	3,483	3,521	3,558	3,596	3,634	3,672	3,710
11	3,748	3,786	3,825	3,863	3,901	3,940	3,978	4,017	4,056	4,094
12	4,183	4,172	4,211	4,250	4,289	4,329	4,368	4,407	4,447	4,486
13	4,525	4,566	4,605	4,645	4,685	4,725	4,765	4,805	4,845	4,886
14	4,920	4,966	5,007	5,047	5,088	5,129	5,169	5,210	5,251	5,292
15	5,333	5,374	5,416	5,457	5,498	5,540	5,581	5,625	5,665	5,706
16	5,748	.....	.....	.....	.....	.....	.....	.....	.....	.....
BASE 100 FT.										
5	1,944	1,985	2,026	2,067	2,108	2,149	2,190	2,231	2,273	2,314
6	2,356	2,397	2,439	2,480	2,522	2,564	2,606	2,648	2,690	2,732
7	2,774	2,816	2,859	2,901	2,944	2,986	3,029	3,071	3,114	3,157
8	3,200	3,243	3,286	3,329	3,372	3,416	3,459	3,502	3,546	3,590
9	3,633	3,677	3,721	3,765	3,809	3,853	3,897	3,941	3,985	4,030
10	4,074	4,119	4,163	4,208	4,252	4,297	4,342	4,387	4,432	4,477
11	4,522	4,567	4,613	4,658	4,703	4,749	4,795	4,840	4,886	4,932
12	4,978	5,024	5,070	5,116	5,162	5,208	5,255	5,301	5,348	5,394
13	5,441	5,487	6,534	5,581	6,628	5,675	5,722	5,769	5,816	5,864
14	5,911	5,958	6,006	6,054	6,101	6,149	6,197	6,245	6,293	6,341
15	6,389	6,437	6,485	6,534	6,582	6,631	6,679	6,728	6,776	6,825
16	6,874	6,923	6,972	7,021	7,070	7,119	7,169	7,218	7,267	7,317
17	7,367	7,416	7,466	7,516	7,566	7,616	7,666	7,716	7,766	7,816
18	7,867	7,917	7,968	8,018	8,069	8,119	.....	.....	.....	.....
BASE 102 FT.										
5	1,981	2,023	2,064	2,106	2,148	2,190	2,232	2,274	2,316	2,358
6	2,400	2,442	2,485	2,527	2,569	2,612	2,655	2,697	2,740	2,783
7	2,826	2,869	2,912	2,955	2,998	3,042	3,085	3,128	3,172	3,216
8	3,259	3,304	3,348	3,392	3,436	3,480	3,524	3,568	3,612	3,656
9	3,700	3,745	3,790	3,835	3,880	3,925	3,970	4,015	4,060	4,105
10	4,150	4,195	4,241	4,286	4,331	4,376	4,422	4,467	4,512	4,568
11	4,603	4,650	4,696	4,742	4,788	4,834	4,881	4,927	4,973	5,020
12	5,067	5,113	5,160	5,207	5,254	5,301	5,348	5,395	5,442	5,490
13	5,537	.....	.....	.....	.....	.....	.....	.....	.....	.....
14	6,017	.....	.....	.....	.....	.....	.....	.....	.....	.....
BASE 110 FT.										
6	2,578	2,623	2,668	2,714	2,759	2,805	2,850	2,896	2,942	2,987
7	3,033	3,079	3,125	3,171	3,218	3,264	3,310	3,357	3,403	3,450
8	3,496	3,543	3,590	3,637	3,684	3,731	3,778	3,825	3,872	3,919
9	3,967	4,014	4,062	4,109	4,157	4,205	4,252	4,300	4,348	4,396
10	4,444	4,493	4,541	4,589	4,638	4,686	4,735	4,783	4,832	4,881
11	4,930	4,979	5,028	5,077	5,126	5,175	5,224	5,274	5,323	5,373
12	5,422	5,472	5,522	5,571	5,621	5,671	5,721	5,771	5,822	5,872
13	5,922	5,973	6,023	6,074	6,124	6,175	6,226	6,277	6,328	6,379
14	6,430	6,481	6,532	6,583	6,635	6,686	6,738	6,789	6,841	6,893
15	6,944	6,996	7,048	7,100	7,152	7,205	7,257	7,309	7,362	7,414
16	7,467	.....	.....	.....	.....	.....	.....	.....	.....	.....
BASE 113 FT.										
5	2,185	2,231	2,276	2,322	2,368	2,414	2,460	2,506	2,552	2,598
6	2,644	2,691	2,737	2,784	2,830	2,877	2,924	2,970	3,017	3,064
7	3,111	3,158	3,205	3,253	3,300	3,347	3,395	3,442	3,490	3,537
8	3,585	3,633	3,681	3,729	3,777	3,825	3,873	3,921	3,970	4,018
9	4,067	4,115	4,164	4,213	4,261	4,310	4,359	4,408	4,457	4,506
10	4,556	4,605	4,654	4,704	4,753	4,803	4,852	4,902	4,952	5,002
11	5,052	5,102	5,152	5,202	5,252	5,303	5,353	5,404	5,454	5,505
12	5,556	5,606	5,657	5,708	5,759	5,810	5,861	5,913	5,964	6,015
13	6,067	6,118	6,170	6,221	6,273	6,325	6,377	6,429	6,481	6,533
14	6,585	6,637	6,690	6,742	6,795	6,847	6,900	6,953	7,005	7,058
15	7,111	7,164	7,217	7,270	7,324	7,377	7,430	7,484	7,537	7,591
16	7,644	7,698	7,752	7,806	7,860	7,914	7,968	8,022	8,076	8,131
17	8,185	8,240	8,294	8,349	8,404	8,458	8,513	8,568	8,623	8,678
18	8,733	8,789	8,844	8,899	8,955	9,010	.....	.....	.....	.....
BASE 120 FT.										
8	3,793	3,843	3,893	3,944	3,995	4,045	4,096	4,147	4,198	4,249
9	4,300	4,351	4,402	4,454	4,505	4,556	4,608	4,660	4,711	4,763
10	4,815	4,867	4,919	4,971	5,023	5,075	5,127	5,180	5,232	5,284
11	5,337	5,390	5,442	5,495	5,548	5,601	5,654	5,707	5,760	5,813
12	5,867	5,920	5,973	6,027	6,081	6,134	6,188	6,242	6,296	6,350
13	6,404	6,458	6,512	6,566	6,621	.....	.....	.....	.....	.....
BASE 122 FT.										
9	4,367	4,419	4,471	4,523	4,575	4,627	4,679	4,732	4,784	4,836
10	4,889	4,942	4,994	5,047	5,100	5,153	5,206	5,259	5,312	5,366
11	5,419	5,472	5,525	5,579	5,632	5,686	5,740	5,794	5,848	5,902
12	5,956	6,010	6,064	6,118	6,172	6,227	6,281	6,336	6,390	6,445
13	6,500	6,555	6,610	6,665	6,720	6,775	6,830	6,886	6,941	6,996
14	7,052	7,107	7,163	7,219	7,275	7,331	7,387	7,443	7,499	7,555
15	7,611	.....	.....	.....	.....	.....	.....	.....	.....	.....
BASE 123 FT.										
5	2,370	2,420	2,169	2,518	2,568	2,618	2,667	2,717	2,767	2,817
6	2,867	2,917	2,967	3,017	3,067	3,118	3,168	3,218	3,269	3,320
7	3,370	3,421	3,472	3,523	3,574	3,625	3,676	3,727	3,779	3,830
8	3,881	3,933	3,985	4,036	4,088	4,140	4,192	4,244	4,296	4,348
9	4,400	4,452	4,505	4,557	1,610	4,662	4,715	4,767	4,820	4,873
10	4,926	4,979	5,032	5,085	5,138	5,192	5,245	5,299	5,352	5,406
11	5,459	5,513	5,567	5,621	5,675	5,729	5,783	5,837	5,891	5,946
12	6,000	6,054	6,109	6,163	6,218	6,273	6,328	6,383	6,438	6,493
13	6,548	6,603	6,659	6,714	6,769	6,825	6,880	6,936	6,992	7,048
14	7,104	.....	.....	.....	.....	.....	.....	.....	.....	.....

TABLE II.—VOLUME OF FILL IN CUBIC YARDS PER 100 LIN. FT. OF LEVEE IN LEVEL SECTION.

	.0	.1	.2	.3	.4	.5	.6	.7	.8	.9
CROWN, 6 FT., SLOPES, 2:1										
0							...	...	22	26
1	30	33	37	41	46	50	55	59	64	69
2	74	79	85	90	96	102	108	114	120	127
3	133	140	147	154	161	169	176	184	191	199
4	207	216	224	233	241	250	259	268	277	287
5	296	306	316	326	336	346	357	367	378	389
6	400	411	423	434	446	457	469	481	494	506
7	519	531	544	557	570	583	597	610	624	638
8	652	666	680	695	709	724	739	754	769	785
9	800	816	831	847	863	880	896	913	929	946
10	963	980						...	.	...
CROWN, 8 FT., SLOPES, 2:1.										
2	89	95	101	107	114	120	127	134	141	148
3	156	163	171	178	186	194	203	211	220	228
4	237	246	255	264	274	283	293	303	313	323
5	333	344	354	[illegible]	376	387	398	410	421	433
6	444	456	468	481	[illegible]	506	518	531	544	557
7	570	584	597	611	625	639	653	667	682	696
8	711	726	741	756	[illegible]	787	803	818	831	850
9	867	883	900	916	933	950	967	984	1,002	1,019
10	1,037	1,055	1,073	1,091	1,109	1,128	1,146	1,165	1,184	1,203
11	1,222	1,242	1,261	1,281	1,300	1,320	1,340	1,361	1,381	1,402
12	[illegible]	1,443	1,464	1,485	1,506	1,528	1,549	1,571	1,593	1,615
13	[illegible]	1,659	1,682	1,704	1,727	1,750	1,773	1,796	1,820	1,843
14	[illegible]	1,890	1,914	1,938	1,963	1,987	2,012	2,036	2,061	2,086
15	2,111	2,136	2,162	2,187	2,213	2,239	2,265	2,291	2,317	2,344
16	2,370	2,397	2,424	2,451	2,478	[illegible]	2,533	2,561	2,588	2,616
CROWN, 12 FT.; SLOPES, 2:1.										
8	830	846	863	879	896	913	930	947	965	982
9	1,000	1,018	1,036	1,054	1,072	1,091	1,109	1,128	1,147	1,166

TABLE II.—Continued.

Cut.	.0	.1	.2	.3	.4	.5	.6	.7	.8	.9
10	1,185	1,205	1,224	1,244	1,263	1,283	1,303	1,324	1,344	1,365
11	1,385	1,406	1,427	1,448	1,469	1,491	1,512	1,534	1,556	1,578
12	1,600	1,622	1,645	1,667	1,690	1,713	1,736	1,759	1,783	1,806
13	1,830	1,853	1,877	1,901	1,926	1,950	1,975	1,999	2,024	2,049
14	2,074	2,099	2,125	2,150	2,176	2,202	2,228	2,254	2,280	2,307
15	2,333	2,360	2,387	2,414	2,441	2,469	2,496	2,524	2,551	2,579
16	2,607	2,636	2,664	2,693	2,721	2,750	2,779	2,808	2,837	2,867
17	2,896	2,926	2,956	2,986	3,016	3,046	3,077	3,107	3,138	3,169
18	3,200	3,231	3,263	3,294	3,326	3,357	3,389	3,421	3,454	3,486
19	3,519	3,551	3,584	3,617	3,650	3,683	3,717	3,750	3,784	3,818
20	3,852	3,886	3,920	3,955	3,989	4,024	4,059	4,094	4,129	4,165
21	4,200	4,236	4,271	4,307	4,343	4,380	4,416	4,453	4,489	4,526
22	4,563	4,600	4,637	4,675	4,712	4,750	4,788	4,826	4,864	4,902
23	4,941	4,979	5,018	5,057	5,096	5,135	5,175	5,214	5,254	5,293
24	5,333	5,373	5,414	.....	.....	.....	.....	.....	.....	.....
CROWN, 8 FT.; SLOPES 3:1.										
8	948	969	990	1,011	1,033	1,055	1,077	1,099	1,121	1,144
9	1,167	1,190	1,213	1,237	1,260	1,284	1,308	1,333	1,357	1,382
10	1,407	1,433	1,458	1,484	1,510	1,536	1,562	1,589	1,616	1,643
11	1,670	1,698	1,726	1,754	1,782	1,810	1,839	1,868	1,897	1,926
12	1,956	1,985	2,015	2,045	2,076	2,106	2,137	2,168	2,200	2,231
13	2,263	2,295	2,327	2,359	2,392	2,425	2,458	2,491	2,525	2,558
14	2,593	2,627	2,661	2,696	2,731	2,765	2,801	2,836	2,872	2,908
15	2,944	2,981	3,017	3,054	3,091	3,129	3,166	3,204	3,242	3,280
16	3,319	3,357	3,396	3,435	3,474	3,514	3,553	3,593	3,634	3,674
17	3,715	3,756	3,797	3,838	3,879	3,921	3,963	4,005	4,048	4,090
18	4,133	4,176	4,220	4,263	4,307	4,351	4,395	4,440	4,484	4,529
19	4,574	4,619	4,665	4,711	4,757	4,803	4,849	4,896	4,943	4,990
20	5,037	5,085	5,132	5,180	5,228	5,277	5,326	5,374	5,423	5,473
CROWN, 6 FT.; SLOPES, 1½:1.										
3	117	122	128	134	140	146	152	158	165	171
4	178	184	191	198	205	212	220	227	235	242
5	250	258	266	274	282	290	299	307	316	325
6	333	342	351	360	370	379	389	398	408	418
7	428	438	448	458	469	479	490	500	511	522
8	533	544	556	567	579	590	602	614	626	638
9	650	662	675	687	700	712	725	738	751	764
10	778	791	805	818	832	846	860	874	888	902
11	917	...	...	...	...	...	...	...	...	...
12	1,067	...	...	...	...	...	...	...	...	...

lining and when the levee has been built in layers or some special provision has been made to prevent settlement, a very thin lining made entirely of cement mortar and reinforced with a wire mesh can be used. If the backing contains adobe or material that may shrink or crack this lining will not be a success. A layer of paper laid all over the surface will prevent the dirt from mixing with the plaster and prevent the earth from drawing away the moisture so fast as to prevent the proper setting of the cement. The cost of such a lining of about ¾ in. thick and including the layer of building paper and the reinforcing mesh will cost about 7 cts. per square foot.

A surface may be also plastered with a cement gun which spreads the plaster evenly by air pressure. It is claimed that this process produces a very impervious plaster with as many as four parts of sand to one part of cement with 10 per cent by volume of the cement of hydrated lime. The cost of this type of lining figures about 15 cts. per square foot for a plaster ½ in. thick. The excessive cost is due to a royalty on a patented machine. Special conditions may justify this high cost however.

ASPHALTUM AND OIL LININGS.

It has been suggested that "Road Oil" might be mixed with the surface earth of a reservoir to form an inexpensive water tight crust. This oil is a heavy by-product gotten by distilling the volatile oils from a heavy crude of asphaltum base. It contains a large percentage of asphaltum and some oils that flow at ordinary temperatures. Road oil will come to the surface on the slope of the reservoir under the action of the sun and when the reservoir is empty and run down to the bottom, thus releasing the earth that was bound in the crust that is now no longer water tight. Road oil is shipped in tank cars and must be heated in the car by steam before it can be gotten out and unless the job is a large one or part of a car can be obtained from a road contractor in the neighborhood, its use may be impracticable. To obviate these difficulties asphaltum is dissolved in a cheap distillate to make a sort of asphaltum paint. Asphaltum is the useful ingredient, the distillate evaporating and leaving it in a finely divided state to bind the particles of sand and earth together. The asphaltum costs more applied in this way but there is the advantage that it can be handled cold and can be shipped and used in small quantities. To get good results, the earth must be dry and contain within certain limits both sand and clay. When the dirt is damp the asphaltum forms a film or coating around a ball or lump which may break up later and leave a hole in the crust which is being made water tight. A natural clayey soil should be mixed with sand and an exceedingly sandy soil is helped by the addition of a little clayey material. The idea is to get the oil onto the surface of the reservoir and let it form a water tight crust and not let it work its way to any great depth as is the case in oiling roads. If a surplus of oil appears on the surface it may be absorbed by the addition of a little sand or road dust. Such a lining is not water tight and will stop only about 50 per cent of the seepage. When 3½ gals. of oil per square yard are used the cost of this lining is about 1¼ cts. per square foot.

If it is possible to obtain bitumen or a prepared asphaltic mixture, a splendid lining may be made by covering the surface of the reservoir with a well rolled layer of crushed rock or coarse gravel and then applying a layer about ¾ in. thick of bitumen or asphaltic mixture (that is asphaltum, rock and sand or gravel and limestone dust). This lining will be water tight for any pressure that may be put upon it in this class of work and will not be affected by any settlement in the bottom or levee surface of the reservoir. A bitumen lining as described above can be put in place for the cost of about 7 cts. per square foot. A good asphaltic mixture called "mastic" contains 9 per cent of asphaltum, 9 per cent of cement or limestone dust, 20 per cent of sand and 62 per cent of gravel that will pass a ½-in. screen. It should be poured and worked at a temperature of 275° F. and if the ground is firm it may be laid without a sub-layer of rock, a layer of paper being used to keep out the dirt. This lining is water tight, elastic, easily repaired, sets like concrete and does not soften up at ordinary temperatures. Without the rock sub-base this lining should cost about 5 cts. per square foot.

All asphaltum linings should be well rolled and the joints between successive pieces of work should be carefully ironed hot to secure a good bond. All cement linings should be kept thoroughly well wet up for a week or ten days after laying to get as even a set as possible and prevent cracks.

Irrigation Pumping Plants in South Africa.

The pumps in most general use in South African irrigation work, according to a consular report, are a reciprocating type, the adaptability of which to varying capacities and heights largely accounts for its popularity. It is specially suitable for installation where the water supply is irregular, demanding no more power than is justified by the work it is called upon to perform. For handling dirty water, however, the centrifugal pump is more satisfactory, owing to its lack of valves; in addition to which its first cost is less, though its effectual capacity is confined within certain height limits—say, 50 feet—and it is most suitable for dealing with a regular supply. In regard to motive power, fuel supply naturally governs the choice of plant in most localities throughout such a country as South Africa. For small pumping installations oil engines up to 6 HP. are very commonly used, and with increasing success now that owners of irrigated farms are less prone than formerly to imagine that the simplicity of working these motors extends to leaving them to run themselves without supervision and without an occasional overhauling and tuning up. For larger installations suction gas plants are extensively employed, one of the most important to date being a 108-brake horsepower equipment at Grahamstown, while others at Oudtshoorn and elsewhere are of nearly that capacity. In some districts anthracite coal is available and cheap to use, but suction gas plants designed for wood fuel or charcoal are more widely in demand. In fact, the suction gas plant is already, and is certain to become more so, the accepted type for irrigation works of any size, and this in spite of the fact that it is easy to find mechanics and drivers possessing an expert acquaintance with steam engines, but with everything to learn about gas engines.

Bridges

Design Features of the Reinforced Concrete Viaduct on Hopple St., Cincinnati, Ohio.

(Staff Article.)

In our issues of Feb. 11 and Feb. 25, 1914, we described and gave design drawings of the principal features of the approaches and superstructure of the reinforced concrete viaduct on Hopple St., Cincinnati, Ohio. In this article we consider the design features of the reinforced concrete piers and footings and give data on the estimated quantities and cost of the viaduct.

III. Substructure.

The character of the subsoil changes considerably along the line of the viaduct. At pier No 2, 75 ft. from pier No. 1 (the east pier), the subsoil consists of strata of yellow-sandy clay, yellow clay, sandy clay, and blue clay, the latter being encountered at a depth of about 15 ft. Near pier No. 12, about 795 ft. west of pier No. 1, the strata consist of blue sandy clay, blue clay, yellow clay, and rock mixed with yellow clay, the latter being found at a depth of about 30 ft. Piers Nos. 16 and 17, which are to be placed in the bed of Mill Creek and are respectively 1,120 ft. and 1,190 ft. west of pier No. 1, will rest on shale which lies only a short distance below the bed of the stream. Between piers Nos. 21 and 22, about 1,510 ft. west of pier No. 1, the strata consist of sandy clay, sand, clay and gravel, and soapstone, the latter being found at a depth of about 35 ft. Shale is found at a somewhat greater depth.

GENERAL FEATURES.

The 26 piers vary considerably in height.

due partly to the profile of the ground and partly to the fact that some of the piers are carried to shale, while others rested on concrete pile foundations. In most cases the footings are carried only a short distance below the surface of the ground, but four of the piers near the west end of the viaduct are carried to a maximum depth of about 50 ft. This great depth is due to the fact that the Baltimore & Ohio Southwestern Railroad Co. expects to build a large freight yard which will necessitate the shifting of Mill Creek several hundred feet west of its present location.

The highest piers, Nos. 16 and 17, are located in the bed of Mill Creek, the distances from the bottom of their footings to the crown of finished roadway surface being about 98 and 97.6 ft., respectively. The next highest pier is No. 23, with a height, from bottom of footing to crown of finished roadway, of about 81.8 ft. This pier is one of a group of four piers of about the same heights whose footings extend distances of from 40 to 50 ft. below the surface, to permit the shifting of Mill Creek. The remainder of the piers are considerably shorter. On account of the angle at which the railroad tracks and Mill Creek cross the line of the viaduct, all of the piers are skewed. Beginning at the east end of the viaduct the angles between the longitudinal axis of each pier and that of the viaduct are: Piers Nos. 1 to 4, inclusive, 77° 53′; No. 5, 68° 01′, and Nos. 6 to 26, inclusive, 59° 20′.

Piers Nos. 1 and 2 have spread reinforced concrete footings resting on yellow sandy clay; piers Nos. 16, 17, 22, 23, 24 and 25 have spread foundations resting on shale; all other piers have concrete pile foundations.

LOADS AND ALLOWABLE STRESSES.

Dead and Live Loads.—In addition to the weight of the superstructure and the piers, the following live loads were used in designing the piers, footings and piling: 150 lbs. per square foot on the full width of the roadway, and 100 lbs. per square foot on the sidewalks.

Allowable Loads on Foundations.—The following loads were allowed on the concrete piles and on the shale and earth: Dead load per pile, 23 tons; dead load plus full live load per pile, 27 tons; maximum load on outside pile when one cantilever only is fully loaded with live load, 35 tons; average bearing on shale 6 tons per square foot; and maximum bearing on earth, 3 tons per square foot.

Allowable Stresses.—The allowable tensile stress in the reinforcing steel of the piers and footings was 20,000 lbs. per square; the allowable compressive stress on the concrete was 700 lbs. per square inch, and the allowable shearing stress on the concrete was 65 lbs. per square inch.

ASSUMPTIONS AS TO THE DESIGN.

Each pier with its adjacent cantilevers was considered a unit, with no bending or arch action transmitted from one pier to another; shear only is transmitted by the joint between the ends of adjacent cantilever arms. Since each pier and its cantilever arms are symmetrical about the center line of the pier, there is no bending due to dead or to full live load on both cantilevers. Piers, footings and piling are designed to withstand the overturning effect produced by the loading of a single cantilever with the full live load.

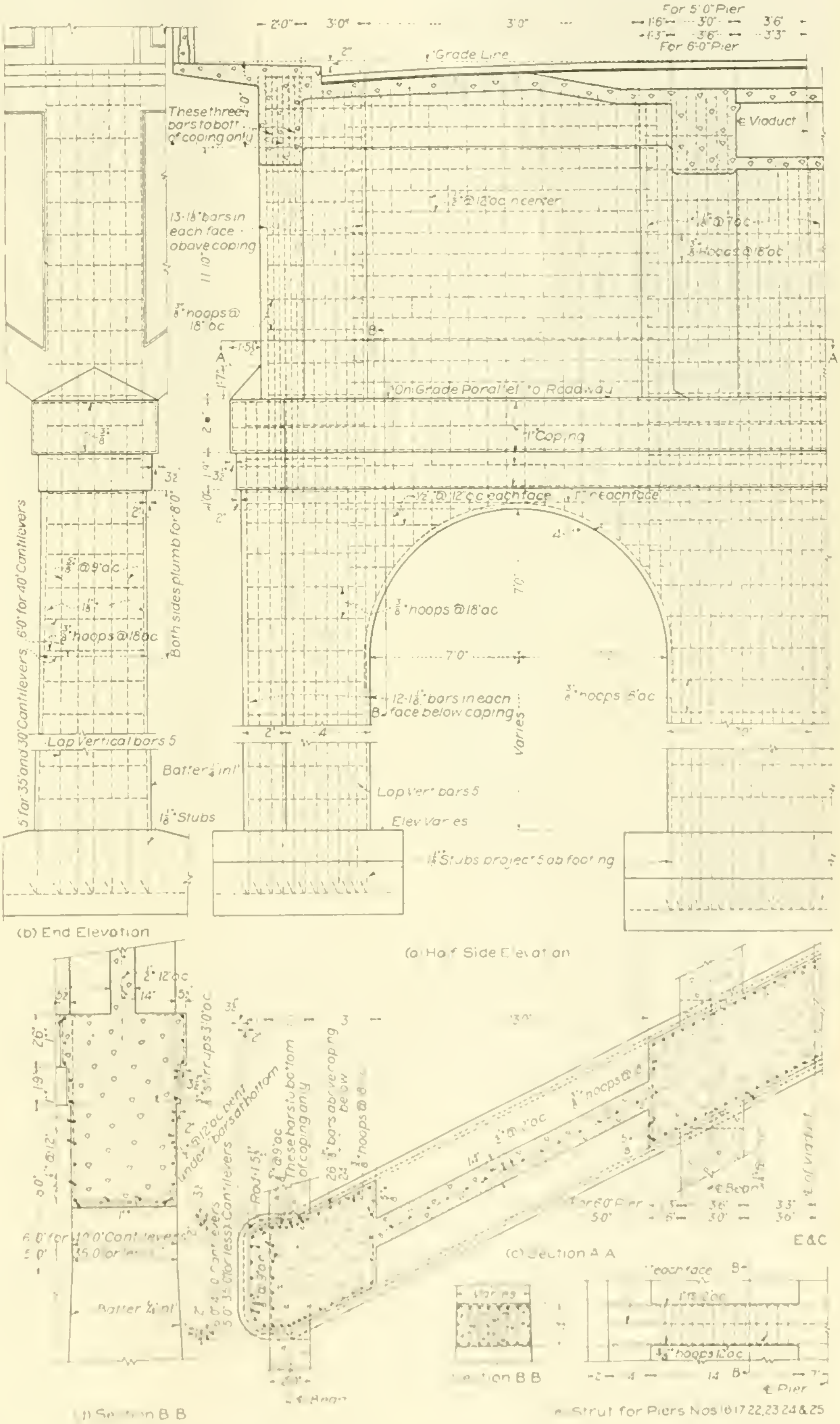
SPECIFICATIONS FOR SUBSTRUCTURE.

The following data were taken from the specifications covering the substructure:

Concrete Piles.—It is planned to use 30-ft concrete piles under the piers, but this length will not necessarily be used. The engineer will specify the length of pile in each case, and the entire length ordered will be paid for. If it is necessary to cut off the pile to bring the top to the proper elevation, no extras will be allowed for this cutting. The contractor must test, at his own expense, twelve piles by loading each pile with a load of 45 tons. The settlement under this load shall not be greater than ½ in., in a period of two weeks.

The contractor was allowed to bid on different kinds of concrete piles. The successful contractor's bid was based on the use of Raymond piles, the specifications for which were: Raymond piles shall have a diameter at the top of 20 ins. and at the bottom of 8 ins., using the 30-ft. pile as a basis. The concrete shall be composed of 1 part Portland cement, 2 parts sand and 4 parts crushed stone or crushed gravel 1 in. and under. The concrete shall be a wet mix which will readily flow. It shall be thoroughly mixed and tamped in the shells as they are filled. The steel shell shall be of No 20 gage. The shells shall be inspected after driving by lowering a light into them, and any which are found to be defective shall be removed.

Piers and Footings.—The concrete in the piers and footings shall be composed of 1 part Portland cement, 2 parts sand and 4 parts crushed stone 1 in. and under. Concrete shall be paid for in place, including all formwork and reinforcing steel.



(b) End Elevation

(a) Half Side Elevation

(c) Section A A

(d) Section B B

(e) Strut for Piers Nos 16 17 22, 23 24 & 25

Fig. 1. Elevations and Sections of Typical Reinforced Concrete Piers of Hopple St. Viaduct, Cincinnati, Ohio. Drawing also Shows Intermediate Strut Used for High Piers.

DESIGN FEATURES OF SUBSTRUCTURE.

Piers.—Figure 1 shows elevations and sections of a typical pier. The drawing also shows changes in dimensions due to differences in the lengths of the cantilever arms, and gives a detail of the intermediate strut which is used to connect the divided pier shafts of the six highest piers. The half elevation shown in Fig. 1 (a) gives the dimensions of the principal details, together with the size and arrangement of the reinforcing rods. It will be noted that the piers have two 14-ft. arched openings, and that each pier shaft is carried on a separate footing. The side shafts are each 6 ft. wide, and the central shaft has a width of 14 ft. Figure 1 (b) shows a side and an end elevation of the pier. It will be noted that the pier has a double coping course. The thickness of the piers which carry 40-ft. cantilevers is 6 ft., for a distance of 8 ft. below the lower coping course; below this the thickness increases at the rate of ¼ in. per foot of height. For the piers which support 30 and 35-ft. cantilevers the thickness of the pier shafts is 5 ft., increasing at the same rate as for the 6-ft. shafts. Figure 1 (c) shows a horizontal section of the pier above the spring line of the curved cantilever ribs. It will be noted that the thickness of the shaft above the spring line is reduced to 11 ins. between the outside and inside ribs, and that it is reinforced with both vertical and horizontal rods, placed at the center of the section. Between the two inside ribs this part of the pier has the full thickness of the shaft below the spring line of the ribs. Figure 1 (d) shows a vertical section through the coping and through a portion of the pier shaft. It will be noted that some of the reinforcement is bent to follow the outline of the pier shaft.

For the two piers Nos. 16 and 17, located in the bed of Mill Creek, and for the four piers Nos. 22 to 25, inclusive, near the west end of the viaduct (the future bed of Mill Creek), the heights are such as to require the bracing of the divided shafts at points midway between the tops of the footings and the tops of the arched openings. Figure 1 (e) shows an elevation and section of the reinforced concrete strut used to connect the outside shafts to the center one, for these piers.

TABLE 1.—DIMENSIONS AND REINFORCEMENT FOR FOOTINGS ON SHALE. (See Fig. 2.)

Footing No.—	Bottom elev., ft.	A. ft. ins.	B. ft. ins.	C. ft. ins.	D. ft. ins.	E. ft. ins.	"y" sq. bars.	"x" sq. bars.
Center Footing Schedule.								
1	64.0	56-0	14-0	3-6	3-6	3-8	¾-in. at 7 ins.	½-in. at 12 ins.
2	62.0	18-0	26-0	3-3	3-3	6-0	1⅛ in. at 6 ins.	½-in. at 12 ins.
16	15.0	17-6	17-0	4-1	4-1	6-0	26, ⅞-in. eq. sp.	½-in. at 12 ins.
17	15.0	17-6	17-0	4-1	4-1	6-0	26, ⅞-in. eq. sp.	½-in. at 12 ins.
22	25.0	16-0	17-0	3-9	3-9	6-4	28, ⅞-in. eq. sp.	½-in. at 12 ins.
23	22.0	18-0	18-0	4-3	4-3	6-5	28, ⅞-in. eq. sp.	½-in. at 12 ins.
24	22.0	18-0	18-0	4-3	4-3	6-5	28, ⅞-in. eq. sp.	½-in. at 12 ins.
25	25.0	18-0	18-0	4-3	4-3	6-5	28, ⅞-in. eq. sp.	½-in. at 12 ins.
North and South Footing Schedules.		a. ft. ins.	b. ft. ins.	c. ft. ins.	d. ft. ins.	e. ft. ins.		
12	62.0	9-0	20-0	3-3	3-6	4-5	1-in. at 6¾ ins.	½-in. at 12 ins.
16	15.0	9-0	14-0	4-1	5-0	4-1	12, ¾-in. eq. sp.	½-in. at 12 ins.
17	15.0	9-0	14-0	4-1	5-0	4-1	12, ¾-in. eq. sp.	½-in. at 12 ins.
22	25.0	8-0	14-0	3-9	4-11	4-8	13, ¾-in. eq. sp.	½-in. at 12 ins.
23	22.0	8-0	16-0	4-3	5-5	5-4	11, ⅞-in. eq. sp.	½-in. at 12 ins.
24	22.0	8-0	16-0	4-3	5-5	5-4	11, ⅞-in. eq. sp.	½-in. at 12 ins.
25	25.0	8-0	16-0	4-3	5-5	5-4	11, ⅞-in. eq. sp.	½-in. at 12 ins.

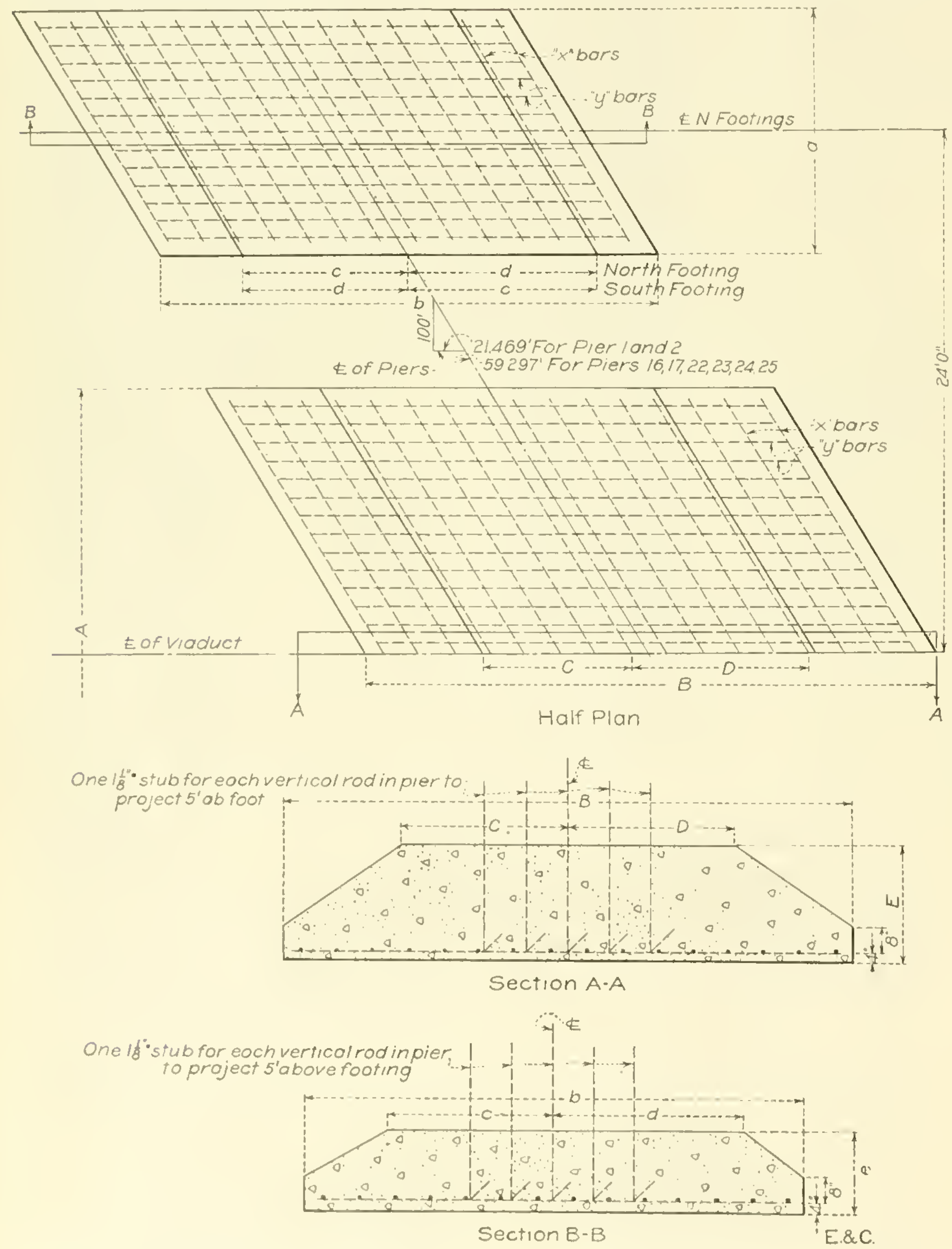


Fig. 2. Plan and Sections of Reinforced Concrete Footings on Shale, Hopple St. Viaduct.

Pier No. 1 (the east pier) is low and has a continuous footing. Piers Nos. 2, 3 and 4 are also comparatively low, but have divided footings. These piers have design features somewhat different from those shown in Fig. 1. The arched opening shown in Fig. 1 is replaced by a flat opening which extends 2 ft. above the top of coping of these piers. Above the opening there is a heavy strut which has a depth of 5 ft. and a thickness equal to that of the pier shaft. Above this strut and between the outside and inside ribs the concrete has a thickness of 14 ins. The shafts for piers Nos. 1 and 4 are 5 ft. thick, and those for piers Nos. 2 and 3 have a thickness of 6 ft.

Footings.—Two types of reinforced concrete footings are used, those supported on shale, and those carried on concrete pile foundations. Figure 2 shows detailed plans and sections of the side and center footings for a typical pier on shale. It will be noted that the stub bars, one for each vertical reinforcing bar of the pier, are hooked around the horizontal bars in the footing and project 5 ft. into the pier shafts. These footings have varying dimensions, depending upon the loading and soil conditions. Table I shows the value of the dimensions which are represented by letters in Fig. 2. The tabulated data also show the size and spacing of the reinforcing bars for each footing, together with the elevation of the bottoms of these footings.

The height of each of the 26 piers, from the bottom of the center footing to the crown of the finished pavement, is given in the following table, No. 1 being the east pier and No. 26 the west one:

Pier No.	Height, ft.	Pier No.	Height, ft.
1	26.40	14	57.84
2	30.43	15	69.23
3	36.59	16	97.62
4	41.62	17	98.01
5	43.37	18	66.40
6	45.13	19	56.51
7	48.02	20	54.62
8	46.04	21	47.73
9	49.20	22	80.84
10	52.36	23	81.81
11	55.52	24	79.65
12	55.97	25	74.49
13	58.42	26	47.33

Figure 3 shows detailed plans and sections for those footings which have concrete pile foundations. It will be noted that the 1⅛-in. stub bars, one for each vertical reinforcing bar, are hook around the horizontal bars of the footings and project into the pier shafts a distance of 5 ft. The 1-in. bars of the concrete piles project into the footing a distance of 2 ft. above the tops of the piles. Table II gives the values of the dimensions represented by letters in Fig. 3, and shows the number of piles required for each pier having a pile foundation, together with the size and arrangement of the horizontal reinforcing bars.

QUANTITIES AND CONTRACT PRICES FOR ENTIRE VIADUCT.

The revised engineer's estimate of quantities and unit and total costs of the various items, together with the successful contractor's bid on each item are given in Table III, on page 310.

The work is undertaken jointly by the Baltimore & Ohio Southwestern Railroad Co.

TABLE II.—DIMENSIONS, NUMBER OF PILES AND REINFORCEMENT FOR FOOTINGS ON CONCRETE PILES. (See Fig. 3.)

Center Footing Schedule.

Footing No.	Bottom elev., ft.	Dimension. A. ft. ins.	B. ft. ins.	C. ft. ins.	D. ft. ins.	E. ft. ins.	No. of piles.	"y" sq. bars.	"x" sq. bars.
3	58.0	18-0	24-0	3-3	3-3	6-9	44	1⅛ ins. at 6¾ ins.	See Fig. 3.
4	55.0	18-0	21-0	3-3	3-3	5-9	37	1 in. at 5½ ins.	
5	55.0	18-0	21-0	3-3	3-3	5-9	36	1 in. at 5½ ins.	
6	55.0	18-0	21-0	3-3	3-3	5-9	39	1 in. at 5½ ins.	
7	54.0	18-0	21-0	3-3	3-3	5-9	39	1 in. at 5½ ins.	
8	58.0	18-0	24-0	3-8	3-8	6-9	46	1 in. at 6 ins.	
9	57.0	18-0	24-0	3-8	3-8	6-9	48	1 in. at 6 ins.	
10	56.0	18-0	24-0	3-8	3-8	6-9	48	1 in. at 6 ins.	
11	55.0	18-0	24-0	3-8	3-8	6-9	48	1 in. at 6 ins.	
12	55.0	18-0	24-0	3-8	3-8	6-9	48	1 in. at 6 ins.	
13	53.0	18-0	24-0	3-8	3-8	6-9	48	1 in. at 6 ins.	
14	54.0	18-0	21-0	3-3	3-3	5-9	42	1 in. at 5½ ins.	
15	43.0	18-0	24-0	3-6	3-6	6-9	46	1 in. at 6 ins.	
18	47.0	18-0	21-0	3-3	3-3	5-9	42	1 in. at 5½ ins.	
19	55.0	18-0	21-0	3-3	3-3	5-9	41	1 in. at 5½ ins.	
20	55.0	18-0	21-0	3-3	3-3	5-9	42	1 in. at 5½ ins.	
21	60.0	18-0	21-0	3-3	3-3	6-9	39	1 in. at 5½ ins.	
26	50.0	18-0	24-0	3-6	3-6	6-9	46	1⅛ ins. at 6¾ ins.	

North and South Footing Schedules.

Footing No.	North.	South.	a. ft. ins.	b. ft. ins.	c. ft. ins.	d. ft. ins.	e. ft. ins.	No. piles N.	S.	"y" sq. bars.	"x" sq. bars.
3	62.0	53.0	9-0	18-0	3-3	3-6	5-3	16	16	1⅛ ins. at 7 ins.	See Fig. 3.
4	60.0	49.0	9-0	18-0	3-3	3-6	5-3	14	15	1 in. at 8 ins.	
5	62.0	49.0	9-0	18-0	3-3	4-5	5-3	14	14	1 in. at 8 ins.	
6	60.0	49.0	9-0	18-0	3-3	4-5	5-3	14	15	1 in. at 8 ins.	
7	60.0	47.0	9-0	18-0	3-3	4-5	5-3	14	15	1 in. at 8 ins.	
8	58.0	58.0	9-0	21-0	3-8	4-10	5-3	17	17	1 in. at 6 ins.	
9	57.0	57.0	9-0	21-0	3-8	4-10	5-3	18	18	1 in. at 6 ins.	
10	56.0	56.0	9-0	21-0	3-8	4-10	5-3	18	18	1 in. at 6 ins.	
11	55.0	55.0	9-0	21-0	3-8	4-10	5-3	18	18	1 in. at 6 ins.	
12	55.0	55.0	9-0	21-0	3-8	4-10	5-3	18	18	1 in. at 6 ins.	
13	53.0	50.0	9-0	21-0	3-8	4-10	5-3	18	18	1 in. at 6 ins.	
14	54.0	54.0	9-0	18-0	3-3	4-5	5-3	16	16	1 in. at 8 ins.	
15	13.0	43.0	9-0	21-0	3-6	4-7	5-3	17	17	1 in. at 6 ins.	
18	47.0	47.0	9-0	18-0	3-3	4-5	5-3	17	17	1 in. at 8 ins.	
19	55.0	55.0	9-0	18-0	3-3	4-5	5-3	16	16	1 in. at 8 ins.	
20	60.0	46.0	9-0	18-0	3-6	4-7	5-3	16	17	1 in. at 8 ins.	
21	65.0	53.0	9-0	18-0	3-3	4-5	5-3	14	15	1 in. at 8 ins.	
26	50.0	50.0	9-0	21-0	3-6	4-7	5-3	17	17	1⅛ ins. at 7 ins.	

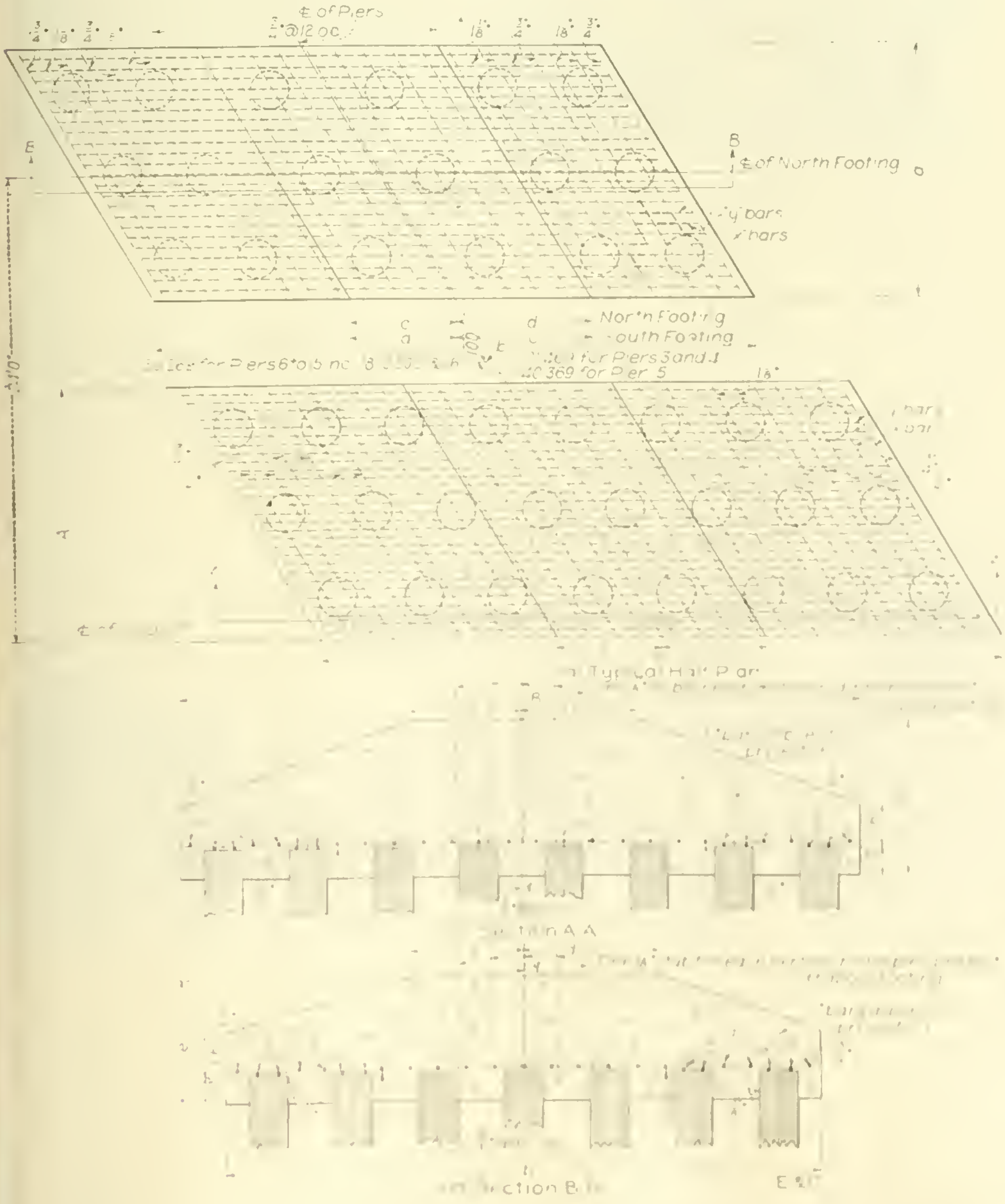


Fig. 3. Plan and Sections of Reinforced Concrete Footings on Concrete Piles, Hopple St. Viaduct.

and the city of Cincinnati, the former assuming one-third and the latter two-thirds of its cost.

PERSONNEL.

The Hopple St. Viaduct was designed under the direction of Mr. H. M. Waite, chief engineer, and Mr. F. L. Raschig, engineer of structures, Department of Public Service of Cincinnati. The construction will be continued under Mr. F. S. Krug, who has succeeded Mr. Waite as chief engineer, and Mr. F. L. Raschig. The Kirchner Construction Co. of Cincinnati has the contract for the construction of this viaduct.

Some Specifications for Reinforced Concrete Highway Bridges and Culverts.

The following recommendations were submitted to the American Concrete Institute, at its recent annual convention in Chicago, by the Committee on Reinforced Concrete Highway Bridges and Culverts. These recommendations were submitted, not as final, but merely as preliminary suggestions, in the hope of bringing out the discussion of doubtful points and information regarding questions which the committee could not answer. The members of the committee were: Willis Whited, chairman, Lemuel Holmes, A. N. Johnson, A. M. Lovis, and Henry H. Quimby.

DEAD LOADS.

The following are given as average weights per cubic foot of the materials mentioned, but if the weights of the materials to be actually used are definitely known to be different from those given, the correct weights should be used:

Materials.	Lbs. per cu. ft.
Earth filling	110
Plain concrete	150
Reinforced concrete	155
Steel	490
Cast iron	450
Vitrified brick	140
Common brick	125
Granite and limestone masonry	165
Sandstone	160
Macadam-Telford	150
Pine, fir, etc.	42
Oak and yellow pine	48
Creosoted timber	60

The weight imposed by earth filling should, in ordinary cases, be considered as including all filling included between vertical planes passing through the faces of the abutments.

If, however, the height of the fill exceeds about two-thirds the distance face to face of the abutments, the live load may be neglected and a very considerable proportion of the weight of the filling considered will be supported by friction between it and the approach filling. The amount of load thus transmitted is greatly affected by the cohesion of the soil, of which there is nearly always more or less. (This paragraph is not approved by Mr. Lemuel Holmes.)

This whole subject requires further investigation

LIVE LOADS.

Class "A" Bridges—Main thoroughfares leading from large towns.

In view of the extensive introduction of the heavy motor trucks and traction engines, and the probable general use of such vehicles in the future, it is recommended that bridges on main thoroughfares and other roads which are likely to be used for heavy hauling, be designed to carry 20-ton trucks, with axles about 10 ft. center to center, 14 tons on rear axle and 6 tons on fore axle; wheels about 5 ft. center to center. Outside of the large cities it is recommended that only one such vehicle be assumed to be on the bridge at any one time, the likelihood of more than one being on the bridge, in a position to produce maximum stresses at the same time, is so remote that this assumption is considered safe. It is advised that such very heavy loads be considered as occupying only the ordinary width of the road, about 8 ft. in width and about 35 ft. in length. Congested traffic of heavily loaded wagons or motor trucks will rarely impose a load of more than 100 lbs.

TABLE III. ENGINEER'S ESTIMATED QUANTITIES AND UNIT PRICES AND SUCCESSFUL CONTRACTOR'S BID FOR VARIOUS ITEMS (HOPPLE ST. VIADUCT.)

Item.	Quantity.	Engineer's estimated price.	Contractor's unit bids.	Engineer's total estimate, each item.	Contractor's total bid, each item.
Trench excavation, earth, cu. yds	11,000	$ 1.50	$ 2.50	$ 16,500	$ 27,500
Trench excavation, rock, cu. yds	2,000	3.00	3.00	6,000	6,000
Raymond concrete piling, lin. ft	43,000	1.00	0.80	43,000	34,400
Reinf. concrete piers and footing, cu. yds	10,000	9.00	9.00	90,000	90,900
Reinf. concrete containing walls, cu. yds	700	15.00	15.00	10,500	10,500
Reinf. concrete superstructure, complete*	......	.....	.....	189,000	188,600
Grading excavation, cu. yds	8,700	0.60	0.60	5,220	5,220
Drain pipe, lin. ft	900	1.00	1.00	900	900
Manholes, each	3	100.00	100.00	300	300
Inlets, each	7	50.00	50.00	350	350
Granite curb, lin. ft	1,000	1.00	1.00	1,000	1,000
Concrete base, cu. yds	500	5.50	4.00	2,750	2,000
Granite pavement, sq. yds	1,400	2.80	2.80	3,920	3,920
Wood block pavement, sq. yds	11,000	2.50	2.30	27,500	25,300
Cement sidewalks, sq. ft	8,500	0.14	0.15	1,190	1,275
Sand blast finish, complete	......	....	....	6,500	6,500
Total contract work	......	..	...	$404,630	$403,765
Field supervision	......	...	...	11,050	10,000
Total contractor's bids	......		...	....	$413,765
Engineering costs	......		...	3,600	....
Advertising costs	......		...	720	....
Engineer's estimate of total cost†	......	....	....	$420,000	....

*Includes all reinforced concrete work above spring line of arched ribs.
†Exclusive of property and damages.

per square foot, over a considerable area. The above mentioned 20-ton truck gives a load of about 140 lbs. per square foot on the area actually occupied, but it is considered extravagant to assume that a large bridge is covered with such heavy loads. A load of 100 lbs. per square foot is thought ample to assume for the loading of spans more than 60 ft. long, in designing the trusses or main girders. It is thought to be safe to reduce this assumed load in the case of longer spans, to the following values:

Length of span, ft.	Assumed load, lbs. per sq. ft.
80	90
100	80
125	75
200 and over	70

With all intermediate spans in proportion.

The greatest load that is liable to be imposed on a bridge sidewalk occurs when there is some excitement in the neighborhood, which attracts a large crowd, and for which the bridge affords an especially good point of view. In that case the crowd forms a compact mass, against the railing, not more than 4 ft. deep, making a load seldom exceeding 100 lbs. per square foot over a very considerable space. The remaining portion of the sidewalk may be covered by a moving crowd which can scarcely weigh more than 40 lbs. per square foot. It may be advisable, sometimes, to so design sidewalk slabs that if a street car or motor truck accidentally gets upon the sidewalk it will not go through. Such accidents are so rare that it is thought safe to allow materials to be stressed somewhat beyond the elastic limit in such cases.

Class "B" Bridges.—Although it is impossible to determine beforehand, especially in the newer parts of the country, whether any given road is to be used for heavy traffic, it seems extravagant, at least in the cases of larger spans, to design bridges to carry much heavier loads than can be expected to come upon them. It is recommended that bridges of this class be designed to carry 15-ton trucks, with axles 10 ft. apart, 5 tons on the front and 10 tons on the rear axle. This will allow for a considerable overloading of existing motor trucks. It is further recommended that only one truck be assumed to be on the bridge at one time, in designing the floor system, that it be assumed to cover a width of 8 ft. and a length of 35 ft. and that the remainder of the bridge is covered with a load of about 90 lbs. per square foot for spans up to 60 ft.

The longer spans, the trusses and main girders should be designed for the following loads:

Length of span, ft.	Assumed load, lbs. per sq. ft.
80	80
100	70
125	65
150	60
200 and over	55

With intermediate spans in proportion.

Sidewalks should be designed to carry the same loads as in the case of Class "A" bridges.

Special Bridges.—City bridges and bridges carrying traffic connected with mines, quarries, lumber regions, mills, manufactories, etc., require special consideration and should of course be designed to carry and load which can reasonably be expected to pass over them, bearing in mind the likelihood of heavy traction engines and motor trucks coming into extensive use in the not distant future.

Stringer Loading.—The maximum stress in a stringer, due to a wheel load, occurs evidently when the wheel is directly over it. It is not thought proper to assume any distribution of the load to adjacent stringers, unless the bottom reinforcement in the slab is made continuous. In that case the distribution is proportional to the relative stiffness of the slab and the stringers, said stiffness being proportional to the moments of inertia multiplied by the modulus of elasticity of materials, and inversely proportional to the cube of the span. In determining this distribution, due account must be taken of the fact that deflection of the slab decreases toward the end of the stringers, and also of the fact that whatever load is carried to the adjacent stringers deflects them also. It is therefore recommended that wherever practicable the bottom reinforcement of slabs be made continuous over the stringers.

Slab Loading.—The distribution in a direction parallel to the supports of a concentrated load resting on a slab, supported at two opposite edges only, evidently depends upon the same principles as those mentioned under "Stringer Loading." The main difference being that what corresponds to the stringer in the former case is of indefinite width in the present case. Adequate theoretical investigations of this question appear to be lacking. For the present it seems fair to assume that the distribution each side of a concentrated load is equal to about one-third the length of the span, and that the cross reinforcement should be designed accordingly, which would require it to have an area of at least one-half of the principal reinforcement. The distribution of a concentrated load through earth filling on the top of a slab does not appear to be very well understood.

Bridges Carrying Electric Cars.—Electric traction is still in its infancy and nobody is able to forecast its future development. It seems probable, however, that it will not be profitable to run cars weighing more than 50 tons each, at a speed that would be permitted on any public road. If very high speeds are desired, the traction company will doubtless be required to operate over its own right-of-way. It is recommended that bridges carrying either urban or interurban electric cars be designed to carry 50-ton cars on two trucks, spaced 30 ft. center to center, each truck having two axles spaced 7 ft. center to center. The committee sees no reason for changing the customary practice of assuming that an axle load is distributed over three ties.

Loading on Arches.—The deflection of an arch being much less than that of a beam of the same length, the method recommended for determining the lateral distribution of a concentrated load over arch sheeting appears to be different from the distribution over flat slabs. It seems doubtful if the distribution in each direction can be greater than twice the thickness of the arch sheeting. This question should be investigated.

Stresses in Arches.—As all arches that are not provided with hinges act as elastic arches until cracks are formed, due to excessive tension at some point or points in the concrete, it is manifestly proper to calculate the stresses in them according to the elastic theory.

As concrete is a very poor conductor of heat, it is not thought necessary in calculating reinforced concrete arches to assume so much variation in temperature as is usual in designing steel structures, although the outside layers of concrete are of nearly the same temperature as the surrounding air, and those layers are stressed more heavily than any of the others. It is thought that an extreme variation of about 80° F. in the northern states is sufficient to allow for, in any case, and that can be reduced if the arch ring is thicker or if there is much earth filling in the spandrels.

BEARING POWER OF PILES.

The usual formula for the safe bearing power of wooden piles is:

$$B = \frac{2\,W\,H}{S + 1}$$

in which

W = weight of hammer in pounds,
H = weight of fall of hammer in feet.
S = penetration in inches per blow, average of last few blows.

If reinforced concrete piles are molded before being driven, the head of the pile is usually cushioned more or less, and the pile is generally much heavier in proportion to the weight of the hammer than is the case with wooden piles. It is recommended that the hammer should be at least as heavy as the pile.

If concrete piles are molded in place, measures should be taken to prevent damage to them by the driving of neighboring piles or otherwise.

BEARING POWER OF SOILS.

This subject is under investigation by a committee from the American Society of Civil Engineers. It may be some years before their final report is submitted and even that will be subject to revision from time to time, as human knowledge is extended. In the meantime the committee would submit the following preliminary table:

Material.	Safe bearing power. tons per sq. ft.
Quicksands and wet soils	0.1 to 1.0
Dry earth, according to depth below surface	1 to 3
Moderately dry clay, confined	2 to 4
Dry stiff clay	4 to 6
Sand confined	2 to 6
Sand compact and cemented	4 to 8
Gravel cemented	8 to 12
Rock	25 to 200

Foundations should be carried down below frost unless they are on rock and thoroughly drained. Soil that contains the roots of plants is generally compressible. Undisturbed soil is much less compressible than filling or similar soil, even though it has been in place many years. The bearing power of sand, gravel and dry clay increases rapidly with the depth below the surface of the ground. None but the smallest structures should be founded on earth filling.

The pressure of earth against abutments, wingwalls and retaining walls varies so widely with the character and condition of the earth in question, that nothing more than a few general suggestions can be given. It is hoped that the committee from the Society of Civil Engineers, which is studying the bearing power of soils, will take up this subject also. In ordinary cases where the filling is well drained its pressure will seldom be more than that of a liquid weighing 25 lbs. per cubic foot, and is frequently much less. The pressure due to land slides is often several times this amount. If the filling is clayey and is allowed to become waterlogged, its lateral pressure is greatly increased. The effect of the freezing of the filling must also be considered.

REQUIRED WATERWAY.

The usual formula for a waterway for culverts is:

$$a = C A^{3/4}$$

in which

a = required area of culvert in square feet,
A = drainage area in acres.
C = a constant, depending on the length and character of the drainage area and may vary from about 0.3 to 2.0 in regions where the mean annual rainfall is about 50 ins.

The capacity of the culvert will be much greater if the wingwalls are flush with the abutments and flare about 30°, the sides and bottom of the culvert are smooth and straight, and the bottom has a good shape.

If reliable information covering a number of years can be obtained regarding the adequacy of the old bridge crossing the same stream, it is much more useful in determining the size of a culvert or bridge [illegible] formula

At Regina, the capital of S[illegible] utility and works committ[illegible] capital expenditure of over [illegible] 1911 Principal items are [illegible] $487,752; disposal works and [illegible] $373,665; street railway ext[illegible] pavements and sidewalks, [illegible] tric light and power, $101,[illegible]

Some Features of the Design and Construction of the City Waterway Bridge at Tacoma, Wash.

(Staff Article.)

The City Waterway Bridge connects the business center of Tacoma, Wash., with the manufacturing district on the tide flats. It occupies the line of 11th St., and consists of an upper and a lower deck. The upper deck has a 50-ft. clear roadway and two 10-ft. clear sidewalks. It extends on a slight up grade from A St. to Cliff Ave., in order to clear the railroad tracks at Cliff Ave., thence on a down grade of from 2½ to 1 per cent over the tracks of the Northern Pacific Ry., across the City Waterway and down to the surface on the north side of the waterway. The lower deck has a 19-ft. 4-in. clear roadway and a 10-ft. sidewalk. It extends on a heavy grade from the top of the hill at Cliff Ave. down to the City Waterway; and then turns back on a lower grade to the level of the railroad tracks and streets on the south side of the Waterway. The north approach consists of about 110 ft. of reinforced concrete retaining wall construction and about 775 ft. of steel viaduct and the south approach consists of a timber trestle. The main portion of the bridge, which crosses the river, comprises two 190-ft. fixed truss spans and a [illegible] vertical lift span which affords a clear channel of 200 ft. When the lift span is down it has a vertical clearance above high tide of 60 ft., and when the span is up the clearance is 135 ft. The range of the tide at the bridge site is about 20 ft.

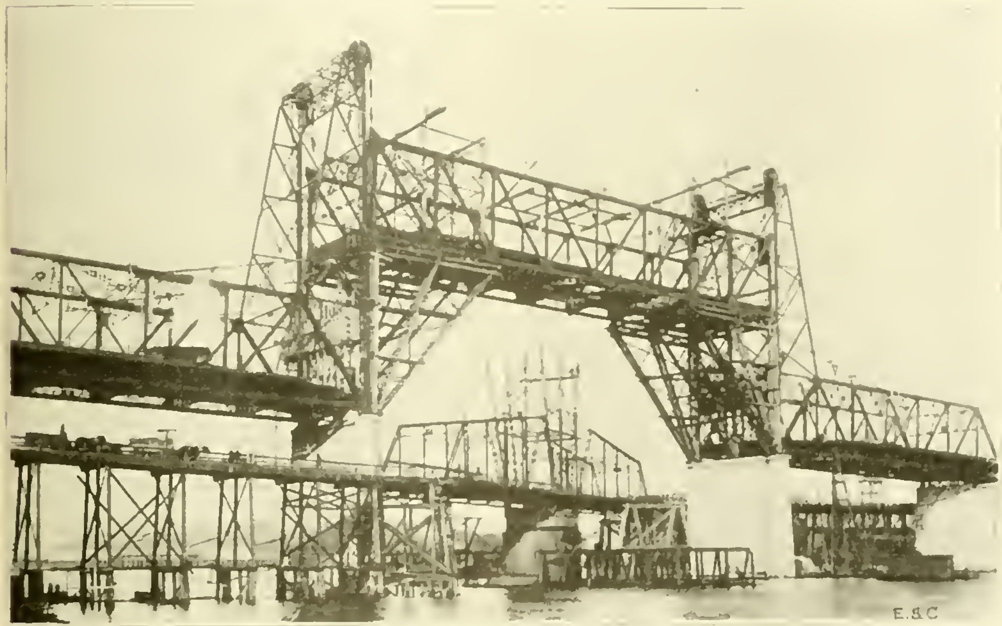


Fig. 1. View of City Waterway Bridge, Tacoma, Wash., Showing Method of Erecting Lift Span—View Also Shows Temporary Bridge With Old Swing Span.

A 16-in. water main is brought up the shore piers, and is carried over the top of the trusses of the fixed spans; thence up the back of the towers and over the lift span on a light truss which was designed to support it.

The upper deck is paved with creosoted wood blocks on creosoted planks, which are in turn supported on creosoted stringers. The surface of the sidewalk is constructed of untreated planks. The roadway carries a double-track street railway in addition to providing for highway traffic. The roadway surface of the lower deck of the south approach consists of untreated 4-in. planks.

DESIGN FEATURES

The piers are built of concrete and rest on pile foundations. The pier shafts consist of frustums of two cones, and are connected throughout their height by reinforced concrete webs. This type of construction results in a considerable saving of material and greatly lessens the load on the pile foundations, as the pier shafts vary in height from 72 to 90 ft. above the bases.

The lift span consists of a simple truss suspended at its four corners by wire ropes, which are carried over cast-steel sheaves 9 ft. in diameter and connect to the concrete counterweights. Each corner of the span is suspended by 16 1½-in. diameter plow steel wire ropes, connected rigidly to the span and to the counterweight through a system of equalizing levers, by which the load is distributed equally over the 16 ropes. The operating machinery is placed in a house located above the center of the lift span. This machinery consists of four drums actuated through a train of gears by two street railway motors, operated by a 500-volt direct current. Each drum carries two ropes, one of which runs underneath a sheave on the corner of the span and connects to the top of the tower; while the other passes from the top of the drum over the same sheave at the corner of the span and connects to the bottom of the tower. All-four drums are similarly connected, and the span is lifted by revolving the drums, which wind up the ropes leading to the tops of the towers, the ropes which lead to the bottom of the towers being paid out at the same time. By reversing the direction of rotation of the drums the span is brought back to its seat.

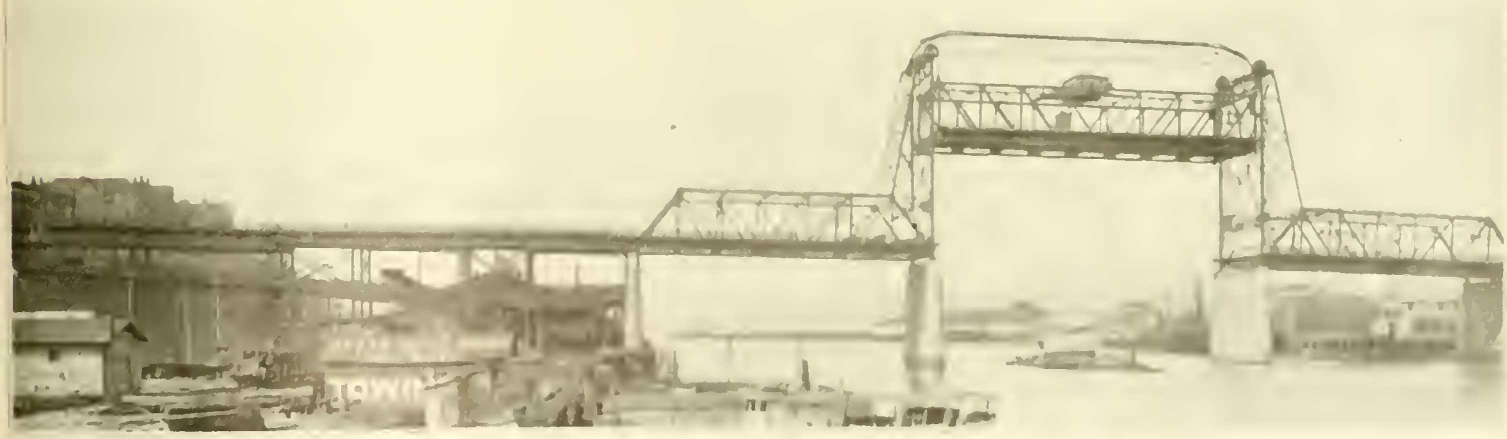


Fig. 2. View of Main Part of Completed City Waterway Bridge, Tacoma, Wash., Showing Type of Bridge and Lift Span in Raised Position.

The motors are operated by a series magnetic controller placed in the operator's cabin at the left of the north sidewalk.

When the span reaches the upper and lower limits of its travel the electric current is automatically cut off and the solenoid brakes are automatically applied. A hand brake is also provided as a safeguard.

In the fixed spans at each end of the lift span there are two electrically operated gates. Two of these gates are lowered and at the same time are swung across to roadway to prevent the traffic from entering the span;

and when the span has been cleared the other gates are closed. These gates are operated by the bridge tender from his cabin.

CONSTRUCTION FEATURES.

The site of this bridge was formerly occupied by a light swing span with a steel approach extending to Cliff Ave. on the south and a wooden trestle extending to the tide flats on the north. It had a much less clearance than the old structure; hence in order to maintain traffic during the construction of the new bridge a timber trestle was built parallel to the old bridge. This trestle extended on the east from Cliff Ave. to the end of the swing bridge and on the west from the north end of the swing span to the tide flats. The old draw span was then swung around to meet these temporary trestles, and traffic was maintained on this structure during the construction of the new bridge (see Fig. 1).

The piers were built by sinking timber cribs into the river bed. The material was then excavated by the open dredging process. The foundation piles were driven inside the cribs to a maximum depth of 115 ft. below low water. These piles were jetted into place, a hammer and two jets being used.

The erection of the viaduct and the two fixed spans presented no unusual features, the fixed spans being erected on falsework.

The methods used in erecting the lift span were somewhat unusual. By referring to Fig. 1 it can be seen that this span was erected on falsework, consisting of two wooden cantilever brackets which rested on the piers and were anchored at their tops to the adjacent spans In this manner the lift span was erected at a sufficient height for the passage of vessels, this span being far enough above the old swing bridge to permit the operation of the latter. The two panels at each end of the lift span were supported by this cantilever falsework. The central panels were erected on falsework, which was constructed on top of the old swing span. Figure 1 shows this falsework being taken down after the erection of the lift span. It was built on the swing span with that span in the closed position. When the center panels were ready to be erected the swing span was opened, and blocking was placed on top of the falsework; the center panels were then erected on this blocking. As soon as the lift span trusses were self-supporting, the falsework on the swing span was removed, and the latter span was again put into service.

The machinery was placed in position and all adjustments were made while the lift span was in its raised position. As soon as this span was completed and paved—it being the last portion of the bridge to be completed—the swing span was again opened, and the center members of the tower were removed so that the lift span could be lowered to its traffic position.

The light truss span, which connects the tops of the towers of the lift span and carries the 16-in. water main, was erected from the lift span when it was in its highest position. Figure 2 shows a view of the main portion of the completed structure, the bridge being finished during the early part of 1913.

QUANTITIES OF MATERIALS AND COST DATA.

The work was let in two lump sum contracts. The cost of the steelwork and machinery, delivered at the bridge site, including the engineering fee, was $236,271. The cost of the substructure and that of erecting the superstructure, including the engineering fee, was $298,900. The total cost of the bridge therefore was $535,171. This cost includes such items as painting, wiring, street car tracks, pavement, machinery houses, gates, and incidental expenses.

The following distribution of the cost of the bridge to the city, together with the quantities and unit costs, are substantially correct:

Superstructure:
  Structural steel in truss spans, 2,560,000 lbs. at 4.1 cts.
  Structural steel in towers, 420,000 lbs. at 4.1 cts.
  Machinery on lift spans and towers, 182,000 lbs. at 13.7 cts.
  Cables and attachments, 42,000 lbs. at 17.5 cts.
  Metal in counterweights, 48,000 lbs. at 4.1 cts.
  Concrete in counterweights, 400 cu. yds. at $9.35.
  Motors and electrical equipment, total cost, $8,000.
  Creosoted timber, at $48 per M.B.M.
  Untreated timber, at $28 per M.B.M.
  Structural steel in approaches, 4 cts. per lb.
Substructure:
  Concrete in pier shafts, 4,300 cu. yds. at $10.00.
  Concrete in pier bases, 6,800 cu. yds. at $17.50.
  Piling for foundations, 33,000 lin. ft. at 40 cts.
  Concrete in pedestals under viaduct, $9.00 per cubic yard.
  Concrete in abutments, $10.00 per cubic yard.

The City Waterway Bridge was built for the city of Tacoma, Wash., Mr. Owen Woods, commissioner of public works, and Mr. W. C. Raleigh, city engineer. The bridge was designed and its construction was supervised by Waddell & Harrington, Kansas City, Mo., to whom we are indebted for the data contained in this article. The machinery and steelwork for the superstructure was furnished by the American Bridge Co. The substructure was built and the superstructure was erected by the International Contract Co., Seattle, Wash.

Sewerage

Features of Design and Financing of the Sanitary Sewerage System and Treatment Works at Vincennes, Indiana.

In November, 1913, a new system of sanitary sewers with arrangements for screening and sterilization were completed at Vincennes, Ind. Some of the details of design and financing are out of the ordinary and are, consequently, of general interest to sewerage engineers. The provision made for sewer ventilation, the design of the devices for sterilization and dispersion through a discharge pipe with multiple outlets in the Wabash River, and the construction by private funds under a municipal purchase agreement, are especially noteworthy. These features were made the subject of a paper by Mr. Robert C. Wheeler before the recent annual meeting of the Indiana Sanitary and Water Supply Association. The major portion of the paper follows:

Vincennes, Ind., is located on the east bank of the Wabash River, about 117 miles southwest of Indianapolis. The topography is very flat and the land slopes gradually downward away from the river. The area inside the city limits is about 2.75 square miles and the total difference in elevation between the extreme high and low points is not over 13 or 14 ft. The population in 1910 was 11,895 and the assessed valuation about $8,200,000.

Vincennes is a railroad center and a good business town, but in 1912, when this work was commenced, it did not have more than four or five miles of sanitary sewers, privately owned, and the greater part of the household waste was taken care of by means of cesspools and privies.

Owing perhaps to the fact that the city is built on a porous deposit of sand and gravel and that the public water supply is taken from the Wabash River and filtered, there has not been a serious epidemic of intestinal disease in recent years.

SEWERAGE SYSTEM.

In 1910, at the request of the State Board of Health, a movement was set on foot to obtain a sanitary sewerage system, and plans and specifications were made for a system to cover the whole city. This plan comprised about 60 miles of sewers, but owing to the difficulties in financing and other reasons, this was cut to about 40 miles, which cover the city very well, so far as it is built up at present. The system as constructed consists of about 40 miles of sanitary sewers, ranging from 8 to 30 ins. in diameter, a pumping station (as it is necessary to pump the sewage during the high stage of the river), sterilization works, and a submerged outlet with multiple orifices into the Wabash River. The pumping station and outlet into the river are about 1,700 ft. below the city limits, on a site which will be suitable for disposal works should further treatment become necessary.

The system is designed to have ample capacity, so that with future extensions it will serve the city when it shall have a population of 30,000, without paralleling existing sewers.

Disposal by Dilution.—The population of the larger towns and cities on the Wabash River watershed, above and including Vincennes, is about 175,000, and the dry weather flow of the Wabash River is approximately 2,000 cu. ft. per second at Vincennes. This is a dilution of 11 to 12 cu. ft. per second for each 1,000 of population located in the towns and cities of the watershed, which presumably will sooner or later have fairly complete sewerage systems. This ratio of dilution is considerably in excess of that legally established in the case of the Chicago Drainage Canal, viz., 3⅓ cu. ft. per second per 1,000 population.

This dilution is ample for the disposal of Vincennes' sewage, but care has been taken to prevent any possible nuisance at the outlet and to protect the people along the river below the outlet in case of an epidemic. There are no large communities for some ten miles below the outfall, and no communities taking water supply from the Wabash for more than thirty miles. Hence the freeing of the sewage from coarser solids and distributing it in the river in such a manner as not to cause local nuisance will probably be sufficient treatment for many years to come.

It seemed advisable, however, as an especial safeguard to the people below in case of an epidemic in Vincennes, to provide apparatus for applying a solution of hypochlorite of lime to the screened sewage when needed.

Screening and Sterilization.—The sewage is passed through two sets of bar screens having clear openings of ⅞ in. and 7/16 in. respectively, to remove the coarser solids which would tend to injure the pumps, clog the distributing outlet, to collect and putrify on the river banks, or be offensive in the flowing stream.

The screen and suction chamber is so proportioned as to allow a detention period of from 10 to 15 minutes or more. This will give time for the hypochlorite to react and will generally allow the sedimentation of any particles which are so large that they would readily clog the orifices in the outlet pipe.

The apparatus for applying the hypochlorite solution consists of a chemical mixing box, two chemical storage tanks with electrically-operated agitators, an orifice box for measuring the amount of solution to be applied, and a perforated pipe for distributing the solution in the screened sewage.

It is proposed to use from 75 to 125 lbs. of the commercial hypochlorite of lime to 1,000,000 gals. of the partially clarified fresh sewage, depending upon the character of the sewage. The screenings will be burned in an incinerator in the pumping station.

Dispersion in River.—The distributing outlet consists of a 24-in. cast-iron pipe, extending into the river about 300 ft. In the last 120 ft. are located five double outlets 6 ins. in diameter and about 30 ft. apart.

The purpose of this type of outlet is, first, to discharge the sewage under the water even in time of extreme low stage, so as to mix the sewage thoroughly with water and prevent nuisance, and second, to conduct the sewage to a point in the river where the current is swift enough to prevent sedimentation and the formation of bars or deposits of sludge along the shore.

Ventilation.—For a number of reasons, including the flooding of some of the streets in time of high stage in the river and the sifting in of gravel at all times, due to the character of the street surfaces, it was not practicable to ventilate the sewers through perforated manhole covers. The method recommended was that of omitting the trap in each main house connection and ventilating the sewers through the main house lateral and its connection to the soil pipe extending above the roof. This omission of a trap on the house connection seems to provide the best method of securing clean and sanitary sewer connections. Each separate connection to the soil pipe in the house should as usual be trapped, and if this is done there is no real reason for a trap in the house connection to the sewer.

Flush Tanks.—A flush tank, with which to keep the sewers clean by flushing them with water about once a day to dislodge particles of suspended matter which may have become deposited on the sides of the sewers, is located at the head of nearly every sewer line. The total number of flush tanks is about 260. Each tank has a capacity of about 250 gals., is provided with a Miller 6-in. automatic siphon, and has a connection to the mains of the water company. The amount of water used is controlled by a flow-regulating device containing a hard rubber disc with an orifice of such a size that under the average pressure in the mains enough water will be discharged into the tank to flush it once in 24 hours.

Flushing of the larger pipes, which will run at only a small fraction of their capacity for the first few years, can be effected by putting temporary stop planks in the manholes, thus backing up the sewage until sufficient head is obtained to produce a velocity to remove deposits.

FINANCING.

The most unusual feature of this work is the method of financing. The ordinary procedure in the construction of sewers in Indiana is in one of three ways.

1. Under an appropriation by council from the tax budget.

2. Under the assessment plan, where the cost is assessed against the property benefited

3. By a bond issue.

The estimated cost of the system was $334,500. This eliminated the first method, which is only practicable for short lengths of sewers such as laterals, etc.

The second method was considered inadvisable or impracticable under local conditions where public opinion was somewhat at variance as to the relative importance of sanitary sewers, storm drains, flood protection street pavements, etc.

The third method would not have provided enough money, since the bonded indebtedness of Vincennes was limited to 2 per cent on an assessed valuation of about $8,200,000 which would only have yielded $164,000.

Finally, all the ordinary methods were abandoned for one reason or another and a new one was adopted. This plan in its final form was as follows:

The common council granted an association incorporated under the laws of the state as the Vincennes Sewer Association a franchise to build, operate, maintain, and extend a system of sewers which was fully described both as to extent and quality by plans and specifications which were made part of the franchise. The city agreed to [illegible] compensation to the sewer association in one of three ways, at the option of the city, as follows:

(a) The city pays an annual rental.

(b) The city purchases the system at a price which decreases each year according to a schedule which is stated in the franchise.

(c) The city allows the sewer association to operate as a public utility, charging for the services rendered according to a schedule included in the franchise.

The first, or rental basis, is the program which is being followed at present and which will probably continue to be followed, unless the city finds it more advantageous to raise the necessary money and purchase the system

The third alternative was introduced to protect the sewer association in case the city did not purchase the system and failed to pay the rental, taxes, insurance, operating expenses, etc. It comes into operation automatically upon the continued failure for 60 days on the part of the city to pay any of the amounts mentioned above.

In the first two cases the cost of maintenance and operation is borne by the city, which also is to keep the system properly insured. In the third case the city assumes no obligation and has no different relation from that with any other public utility, maintenance and operation being borne by the sewer association.

With this franchise as a basis, bonds were issued by the sewer association and were signed by the Mayor merely to identify the bonds as being of the issue which they were represented to be.

The bonds draw interest at 6 per cent and the annual rental is such that it will pay the interest and retire all the bonds in 25 years. Thus, the selling price decreases each year until at the end of 25 years it is equal approximately to the rental due at that time. Upon the payment of this sum the title to the system passes to the city without further payment.

The annual rental is $26,425, to be paid in equal semi-annual installments dating from Jan. 1, 1912, except that the first payment on July 1, 1912, should be $8,500, instead of $13,212.50, provided the system were half completed by that date, or if it were not half completed there would be due on July 1, 1912, so much of $8,500 as the amount of work done was of 50 per cent of the entire system, the remainder of the $8,500 being retained until 50 per cent of the work was completed.

The city has the right to purchase the system after 30 days' notice in writing to the sewer company at the following respective prices:

Year.	Price.	Year.	Price.
Jan. 1, 1914	$334,675	Jan. 1, 1926	$215,200
" 1915	328,000	" 1927	200,900
" 1916	320,350	" 1928	186,100
" 1917	312,625	" 1929	169,650
" 1918	303,925	" 1930	152,750
" 1919	295,200	" 1931	134,275
" 1920	285,475	" 1932	115,350
" 1921	275,725	" 1933	94,850
" 1922	264,975	" 1934	73,300
" 1923	253,700	" 1935	50,250
" 1924	241,400	" 1936	25,650
" 1925	229,100	" 1937	13,000

From this tabulation it appears that the payment of $13,000 on Jan. 1, 1937, gives the city the title to the system.

Assuming maintenance, pumping, operation, insurance, etc., to be $5,000 per year, rental $26,125, annual cost will be $31,125.

On the basis of the present population of 15,000 and an annual cost of $31,125 the per capita annual cost will be a trifle over $2. The cost includes not only sewer service but also an amount to be applied to the purchasing of the system. That is to say, that at a cost of a little more than $2 per capita annually for 25 years the city may secure sewer service and at the end of that time own the system in a well-maintained condition and of a size and extent such that it will not have to be paralleled while the population is doubling in amount.

PRECAUTIONS AND CONCLUSIONS.

So far as I know this is the first sewer system in Indiana to be built under this method of financing, but the results on the whole have been and promise to remain very satisfactory.

Especial care should be taken in drawing the plans and specifications when proceeding under this method to give them proper flexibility and to define clearly the exact position of the city as regards inspection, engineering and overseeing construction.

In the Vincennes work the city's only representative on the ground was the resident engineer, acting under the Board of Public Works. His duties were to act as advisor to the board in regard to suggestions as to changes in the plans, to have general oversight of the work and pass on materials and workmanship, and to make a final inspection of the system and recommend its acceptance or rejection by the Board of Public Works.

Estimates of cost upon which payments during construction were based were made by Mr. George W. Sturtevant, of Chicago, acting as engineer for the sewer association.

The contractor furnished his own lines and grades according to the contract drawings or supplementary drawings furnished by the resident engineer, and at the completion of the work furnished the city with record plans of the system as constructed.

While this arrangement was reasonably satisfactory at Vincennes, it would probably be advisable as a rule for the city to have more complete control by furnishing lines and grades, keeping all records, making estimates of cost, and maintaining more thorough inspection.

With these points carefully considered, this method of procedure may help to solve the problem for other towns where otherwise a lack of funds may prohibit the construction of the whole sewer system at once, and where, as at Vincennes, a relatively expensive outfall may put too heavy a burden on the first section to permit proceeding by sections under the assessment plan.

The plans and specifications were drawn by Hering & Fuller and the work was superintended by George W. Fuller, Robert C. Wheeler being Resident Engineer. The general contractors were Stewart Sheets & Co. Harry Watts, Engineer for the contractor, gave lines and grades and drew the record plans.

A Regulating Device for Insuring Uniform Distribution of Tank Effluent Over Sprinkling Filters, in Successful Use at Pennypack Creek Plant, Philadelphia.

A tapered dosing tank is usually employed for applying settling tank effluent to percolating filters with fixed sprinkler nozzles. The aim of the designer, in proportioning this dosing tank, is to shape it so as to cause a uniform distribution of the sewage over the surface of the beds. The shape of the tank must be calculated from data of very doubtful dependability and when constructed can be altered only at large expense. In the Spring Garden sewage treatment experiment station in Philadelphia a sprinkling filter was operated from a constant head tank and the distribution was controlled by a butterfly valve the rate of movement of which was governed by a cam driven by a water wheel using settled sewage, under a sufficient head, for power. By successive trials the shape of this cam was so made that practically every part of the bed was dosed at a uniform rate. The principle of this experimental apparatus was developed into a machine for the Pennypack Creek sewage disposal works which is now in successful operation. The machine is here described, and the results of its operation are shown, from information contained in the latest annual report of John [illegible] Allen, assistant, under Mr. W. L. Stevenson, Assistant Engineer, of the Sewage Disposal Division of the Bureau of Surveys.

Before taking up a detailed description of the regulating machinery a brief description of the Pennypack Creek plant will be given. The plant consists of two Imhoff tanks for removing the settleable solids and digestion of the resulting sludge; a constant head tank; a regulating house containing the machine for regulating the head upon the nozzle of an acre bed of percolating filters, the effluent of which after being disinfected, passes through a settling basin and over a weir for measurement. The final effluent discharges into the Pennypack Creek near its junction with the Delaware River. The Imhoff tanks were probably the first full-sized tanks of this type designed in this country.

DESIGN AND OPERATION OF REGULATING MACHINE.

Butterfly Valve.—The effective nozzle pressure and therefore the spray from the nozzles of the percolating filters is controlled by the action of a butterfly valve in the feed line between the constant head tank and the filter.

This valve consists of a circular bronze leaf of the diameter of the pipe in which it operates, pivoted centrally on a vertical brass shaft. It has no seat and is prevented from making a complete revolution by exterior stops at full open and closed positions. The bottom of the shaft turns in a socket cast in the pipe section and drilled to fit the shaft. At the top the shaft passes through an ordinary stuffing box and terminates at a bevel gear.

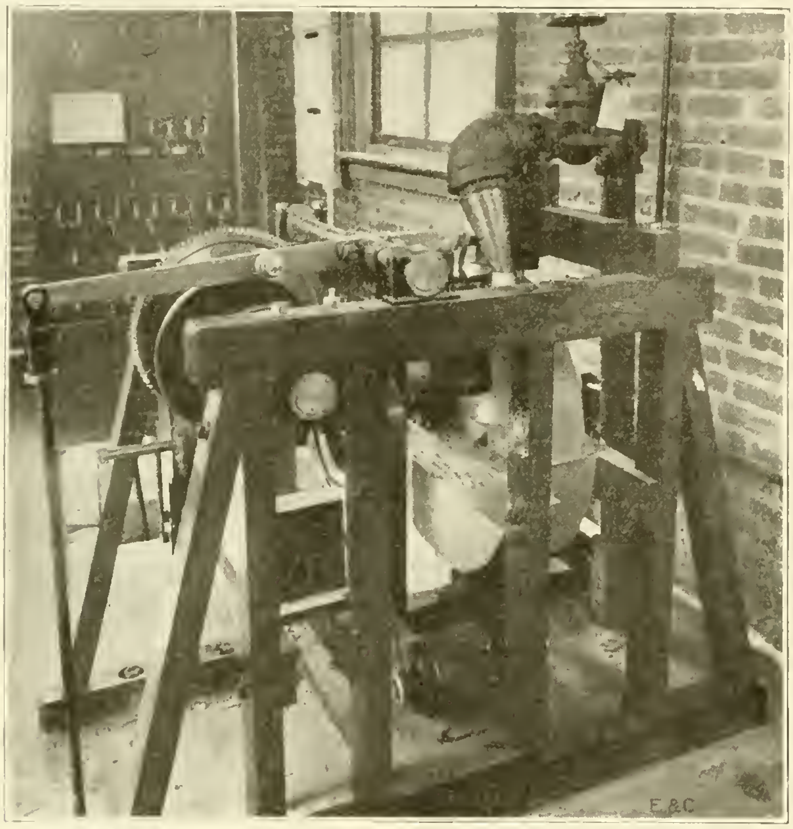


Fig. 1. View of Machine for Regulating Rate of Application and Distribution of Settling Tank Effluent to Surface of Sprinkling Filter Beds, Pennypack Creek Plant, Philadelphia, Pa.

The weight of the shaft and leaf is taken by a ball-bearing beneath the gear, resting on a frame which is adjustable vertically to center accurately the leaf within the pipe. A small by-pass, controlled by a hand valve, is connected around the leaf to maintain the water level in the filters during a prolonged shut down of the butterfly valve.

The bevel gear on the vertical shaft is in mesh with another one on a horizontal shaft crossing above the valve, carried in boxes supported on brackets which are bolted directly to the pipe section. To this horizontal shaft is attached the levers for operating the valve.

The bronze leaf has no sharp edges and the surfaces are curved to oppose a minimum resistance to the flow. It is absolutely balanced and will maintain itself in any position

Operating Machine.—The action of the butterfly valve is controlled by a machine driven by the sewage. This machine is illustrated in Fig. 1. The constant head tank is tapped by a pipe which conveys a small amount of clarified sewage to the machine, where it passes through a combination impulse and gravity water wheel and is then deposited on the surface of the percolating filters. This water wheel transmits its motion to a shaft which drives the main shaft of the machine by means of a worm and worm wheel. An emergency drive is provided in an electric motor.

Both water wheel and motor are connected to the driving shaft by centifugal friction clutches which grip when a certain speed is attained, so that each attains considerable power before being called on to do any work. The motor is thrown into service automatically when the water wheel stops or slows down from any cause, and, as the stopping or slowing down of either one releases its friction clutch, the one in service does not have to carry the inactive one with it.

The main shaft has three cams, one of which is fixed and two are loose on the shaft. The loose cams have spring clutches which engage with the shaft and are held open by arms actuated by electro-magnets, which in turn are controlled by float switches at the constant head tank. These cams transmit a motion, in accordance with their particular outlines, to a lever which is connected directly to the operating shaft of the butterfly valve

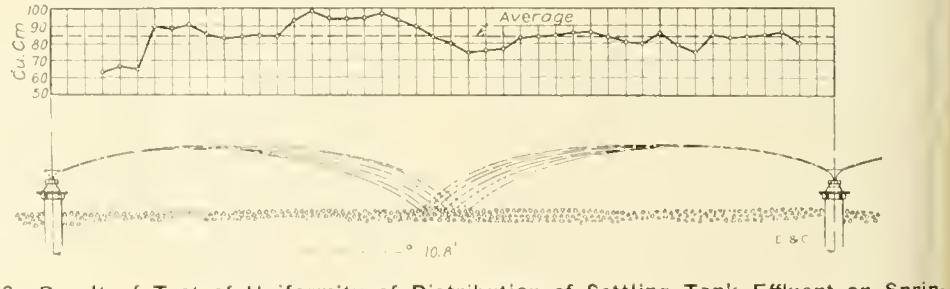


Fig. 2. Result of Test of Uniformity of Distribution of Settling Tank Effluent on Sprinkling Filters at Pennypack Creek Plant.

(The upper diagram shows the amount of sewage, in cubic centimeters, falling upon the surface of tne filter, at the points shown, during twelve displays of the nozzles. The lower diagram shows a cross-section of the filter between two nozzles, and shows the overlap of the spray at maximum head.)

The cams are so shaped as to give a uniform distribution of the spray from the nozzles to full play and return. The difference in the shape of the two cams is that one has a longer period of spray at full head than the other, and therefore is used when sewage reaches the works at times of maximum flow.

The fixed cam, with its lever, operates a counterweight which closes the valve in conformity with the return side of the cam.

The electro-magnets are connected with two-way gravity throw switches at the constant head tank, operated by a ball float and so adjusted as to maintain a constant water level by keeping in service the cam, which is properly adapted to the rate of incoming sewage. When no sewage is entering the tank, the machine is running idle, the cam grips being held open by the magnet arms. As sewage enters and the water level tends to rise in the constant head tank, the float causes the magnets to operate and the short play cam comes into action. If this is not sufficient to care for the quantity, the large cam is thrown in by the magnets. If the level continues to rise, an alarm gong is rung, giving notice to the attendant that additional filter bed capacity is required. When the water level falls, the operations are reversed until the machine is again running idle.

The magnets can be operated either from the line or a storage battery, a mechanical make-and-break contact being provided on the main shaft, so that current is sent through the circuit for an instant only during each revolution of the cams.

An adjustment on the valve operating lever provides the varying opening of the butterfly to compensate for the changing filter area and maintains the radius of action of the spray nozzles under all conditions.

The machine is ball-bearing throughout and requires about 1/16 HP. to operate.

Distribution of Effluent on Beds.—When nozzles were first used for distributing sewage upon percolating filters, they were operated with a fixed hydrostatic head. The sewage was therefore mostly applied to a ring around each nozzle while the other portions of the filter were very scantily dosed. Thus a part of the bed was much overworked and a part was almost unused. Under these conditions the net rate at which the filter could be operated to yield satisfactory effluents was necessarily low.

The use of a dosing tank which alternately filled and emptied its contents through the fixed nozzles caused the spray to cover much more of the surface of the filter, but examinations made of filters so dosed still showed that portions of the surface were much overdosed and others under-dosed.

The ideal distribution on a percolating filter would be obtained when to every portion of the surface of the filter there was applied the same quantity of sewage at a given time. This would result in the use of much higher net rates upon the filter, more uniform purification and decreased areas needed, and as the cost of construction of a percolating filter and the cost of broken stone to fill it is very considerable, any device which will accomplish this uniform distribution and consequent reduction of area will increase its efficiency and will result in large economies.

By means of the cam in the above described machine it has been found possible at the Pennypack Creek Sewage Disposal Works to obtain practically perfect distribution of the sewage upon the surface of the filters, as shown in the accompanying diagram, Fig. 2; a result which has not an equal in any plant in this country where nozzle distributors are used.

DISINFECTION.

The usual type of disinfection apparatus for sewage effluents has many features which make it both hard to operate and inefficient; among these may be mentioned: the carrying of bleaching powder to an upper story, the irritating dust present whenever the solution is made, and the waste of the disinfectant due to the slowness of the diffusion of the solution with the sewage. In the design of the disinfection apparatus of this plant the following devices were used to overcome these features:

The mixing tank, instead of being placed above the solution tanks, as is the usual practice, is placed on the floor and tightly covered. The irritating dust from the bleaching powder will be removed from the building by an exhaust fan and the clear bleaching solution will be diluted with water before being added to the effluent of the percolating filters from a length of lead pipe perforated with very small holes to effect diffusion.

FLOW RECORDING DEVICE.

The device for measuring and recording the rate of flow through the works embodies some features novel in this country. The sewage flows over a "V" notch weir, the upstream side of which is connected to a well, in which a float operates an electrical device which transmits the motion of the float to the operating house where the recording device is located. The operator can see on a dial the rate of flow through the works and upon a clock driven drum the permanent record is made for filing. The device is in use in the sewers of Paris and the recording part of the machine has been used for many years by the Philadelphia Bureau of Surveys in the combined sewers of the city to record storm flows.

The record also serves as an admirable check on the faithfulness of the engineers at the pumping station in the discharge of their duties, and from it may also be obtained the total pumpage.

Notes on the Collection and Disposal of Sewage at Paris, France.

The appended interesting notes on the collection and disposal of the sewage of Paris were brought to our attention by Mr. John W. Alvord, consulting engineer, Chicago, Ill. The information given was compiled by our Consular Agent at Paris, Mr. Robert B. Douglas, and was obtained from him by Mr. H. H. Walker of Chicago during his visit to Paris last summer.

As early as 1861, experiments were made at Paris in the purification of sewer water by the soil. The "tout a l' egout" (everything to the sewer) system is fast superseding the old cesspool and tinette (metal drums) methods, and is now obligatory in the case of new streets. There are at present 750 miles of sewers in Paris. All these sewers run into "collectors," of which there are about 63 miles in the city and 5 miles or so outside the walls. They vary in size from 16 ft. in diameter upwards, the largest galleries being 32½ ft. wide by 14½ ft. high. The water flows down a trough in the center of the gallery and there is a fairly wide walk on either side for the workmen. On one of the large collectors there are boats, one of which is used for the conveyance of visitors, the others are employed as dredges. In another collector electric trolley cars over the central canal are used for the same purposes.

With the exception of a few collectors in the higher parts of the city, all the water is taken to Clichy, where there are extensive works on the banks of the Seine. The water passes through grids which arrest a great portion of the solid matter. About 15,000,000 gals. of mud are taken out at Colombes annually, and rather more at Clichy. The remainder is pumped by powerful machines, capable of pumping 400 to 500 gals. per second to another station at Colombes, where it flows into six large reservoirs, where it remains until the solid matter it still contains has settled to the bottom. This mud is lifted by box dredges lowered from an overhead traveling platform, worked by electricity and dropped into trucks, which carry it to boats which convey it to a depot on the river, where it remains until it is dry, when it is sold for manure. As the supply of this material is considerably greater than the demand, a machine has been invented and is now in course of construction in which the mud will be dried by hot air, and then burned.

The sewage water, freed from solid matters, is pumped by 18 powerful engines, each of 300 or more horsepower. There are also dynamos as supplementary power in case any of the steam engines are temporarily out of order. The sewage is pumped into pipes which convey it to the sewage farms. There are four of these farms, viz.:

	Acres.
Gennevilliers	2,011
Acheres	3,350
Mery-Pierrelaye	1,967
Carrieres Triel	2,347
Total	12,675

The farm at Acheres, which was the one inspected, is a strip of land 6¼ miles long by 1,100 yds. wide, on the left bank of the Seine, bordering one of the many windings which this tortuous river takes. The water which has been pumped from Colombes to a point on the oposite bank of the river, about 160 ft. above the level of the Seine, descends through a syphon under the bed of the river, and is carried through a network of pipes laid in or around all the fields. Each pipe has a screw tap, connected with which is a circular stone basin with two or three openings, by which, when the tap is open the water can be directed into the gutters or troughs which are cut at frequent intervals across the fields. The amount of irrigation (which has been fixed by a committee of experts) must not exceed 3,625,000 gals. a year per acre.

The effect of the sewage water on the vegetation is very remarkable, the growth of the vegetables raised being luxuriant, or almost rank. After watering the vegetables, the water percolates through a sandy soil to drains which are laid 10 ft. under the surface, and these drains lead to small canals which lead to the Seine.

The water which, when it flows in the fields, is of a dark gray or almost black color, is, after it has filtered through the bed of sand, of a crystalline transparency. The workmen and farm laborers drink this without feeling any ill effects; in fact, they are said to be remarkably healthy.

The constrtiction of the sewers of Paris has cost, up to the present time, $40,000,000, to which sum must be added $10,000,000 for the rental of land. The city of Paris was lately empowered by the Government to raise a loan of $180,000,000, out of which $25,000,000 will be devoted to extending the present sewer system and purifying the Seine. The total cost for the upkeep during the year 1911 (the latest year for which statistics are available) was $898,571.

Paris being somewhat of the form of a shallow cup, with the higher ground in the north and south, pumping operations are required at several points of the city to raise the sewage water to a higher level. There are several of these pumping stations; at some of them the engines are worked by "poor gas"; that is to say, by the further extraction of gas from gas coke, which is effected in furnaces or ovens built inside the pumping station. At one place the pumping is effected automatically by compressed air. The water which flows in the lower sewer falls into a well and when it reaches a certain level raises a float which turns on a tap of compressed air, which forces the water into another sewer 15 ft. higher.

It should be mentioned in conclusion that in the larger galleries there is room for the conduits of drinking and Seine water (which latter is used for washing the streets) and which are conveyed in pipes of 30-in. diameter placed along the arch of the gallery. There are also pipes in these galleries for the telephone wires and compressed air, but the gas pipes and electric cables for lighting are excluded in order to avoid any danger from explosions.

Roads and Streets

Methods and Apparatus for Determining Effect of Climate on Road Surfaces, Scotch Experimental Roads.

Experimental roads are planned in ten different districts in Scotland to determine the effect on road crusts of climatic conditions acting in conjunction with traffic. The experiments are to be prosecuted under the direction of the Scottish District of the Institution of Municipal and County Engineers working through a special Road Committee. This committee has reported the kinds of road to be tested and prepared plans for their construction and for prosecuting the tests. From the plans and directions as reported in the "Surveyor" we take the matter relating particularly to the meteorological conditions which alone are unique and unfamiliar to American experimental road practice.

The plan of the tests is to so build the roads as to have all conditions [illegible] the meteorological conditions. The kinds of roads to be built are:

(1) Ordinary water-bound macadam with metal as generally used in the locality, preferably whinstone.

(2) The above, surface tarred with distilled tar to Road Board specification.

(3) Macadam, 3 ins. thick, grouted with a mixture of pitch and sand.

(4) Macadam, 3 ins. thick, grouted with pitch alone.

(5) The above surface sealed with prepared bituminous mixture.

(6) Macadam, superimposed upon and rolled into a layer of tarred chippings, then grouted and surface sealed as above.

(7) Tar macadam, i. e., tarred material surface sealed with distilled tar.

(8) The above, surface sealed with prepared bituminous mixture.

Specifications are given for the construction of each class of road and rules are also given for recording and reporting the test results as they appear; these we omit as not being unusual in American test road practice. The directions for meteorological observations are, however, presented in some detail as follows:

Before any experimental lengths of road are put down the meteorological instruments should be installed. Observations are to be made and recorded of the effects of rainfall, atmospheric humidity, maximum and minimum shade temperatures, solar and terrestrial radiation, winds, fogs, mists and extremes of temperature on the care of the road.

Observation Station.—Figure 1 shows in plan and elevation, the instruments as they are to be set up at the meteorological station. Figure 2 shows the surface box and tubes to be placed in the roadway for the reception of thermometers to record road core temperatures. It is proposed to install at the station a 5-in. Snowdon rain gage, a Stevenson thermometer screen, a wet and dry bulb hydrometer, a self-registering maximum thermometer and a minimum thermometer.

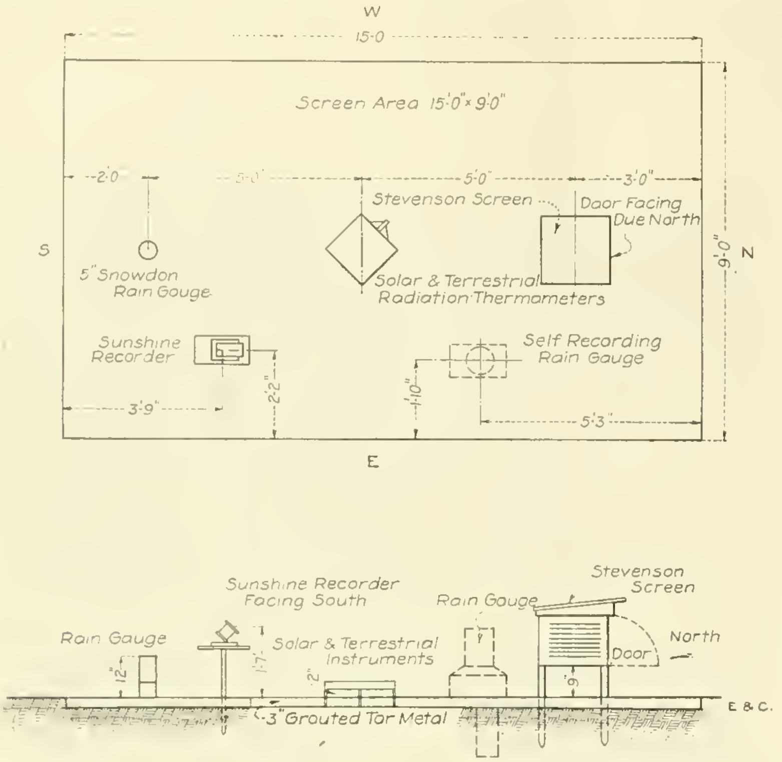


Fig. 1. Arrangement of Meteorological Station for Road Experiments in Scotland.

The observing station should be placed as near to the side of the experimental road as practicable and not more than ¼ mile therefrom. The area 15 x 9 ft., upon which the instruments are to be set up is shown by Fig. 1. It is to be constructed of tar-macadam or pitch-grouted metal 3 ins. in thickness. The level of this area should be, as nearly as possible, the same as the surface of the experimental road in order that the stratum of air over the station may be the same as that over the roadway.

The rain gage should be set exactly 1 ft. above the ground level in an open situation on a level piece of ground free from air eddies and currents. If a perfectly open site cannot be obtained, shelter is the least harmful on the northwest, north and east. The gage should be distant from the object forming the shelter at least twice the height of the object. These conditions should be kept in view when fixing the site for the station. The mode of reading the gage does not require description, except that it and all the other instruments should be read at 9 a. m. each morning, and the readings entered in the record under the previous day's date. In the event of a snowfall, the snow in the funnel should be melted by adding a measured quantity of hot water, and this amount deducted from the gross reading. Approximately 1 ft. of snow is equivalent to 1 in. of rain. The Stevenson screen should be set up so that the door faces north. For meteorological observations the general height of the thermometer is 4 ft., but for road work the temperature should be observed as near the level of the experimental road as possible.

The terrestrial radiation thermometer and solar radiation thermometer are to be placed inside a specially constructed metal box which can be supplied by Messrs. R. & A. Main, Gothic Iron Works, Falkirk, at a cost of about 25s. ($6.07). The box which is shown by Fig. 3 is to be set into the area previously referred to, and in the position shown by Fig. 1. The thermometers are placed in the tubes provided for the purpose. The terrestrial radiation thermometer (spirit filled) is to measure the minimum temperature near the level of the road surface, and the solar radiation thermometer (mercury filled, scale reading to 160° F.) is to measure the amount of solar heat near the level of the road surface. The reading of the solar radiation thermometer is to be entered in the proper column in the record, under the date previous to that of observation, as in the case of rainfall, and maximum shade temperature and the reading of the maximum thermometer in the shade deducted from it and entered in the proper column. The difference so obtained is the measure of solar radiation for the day. This measurement however does not give the duration of maximum solar heat, and an effort is to be made to get a measure of this by installing a sunshine recorder.

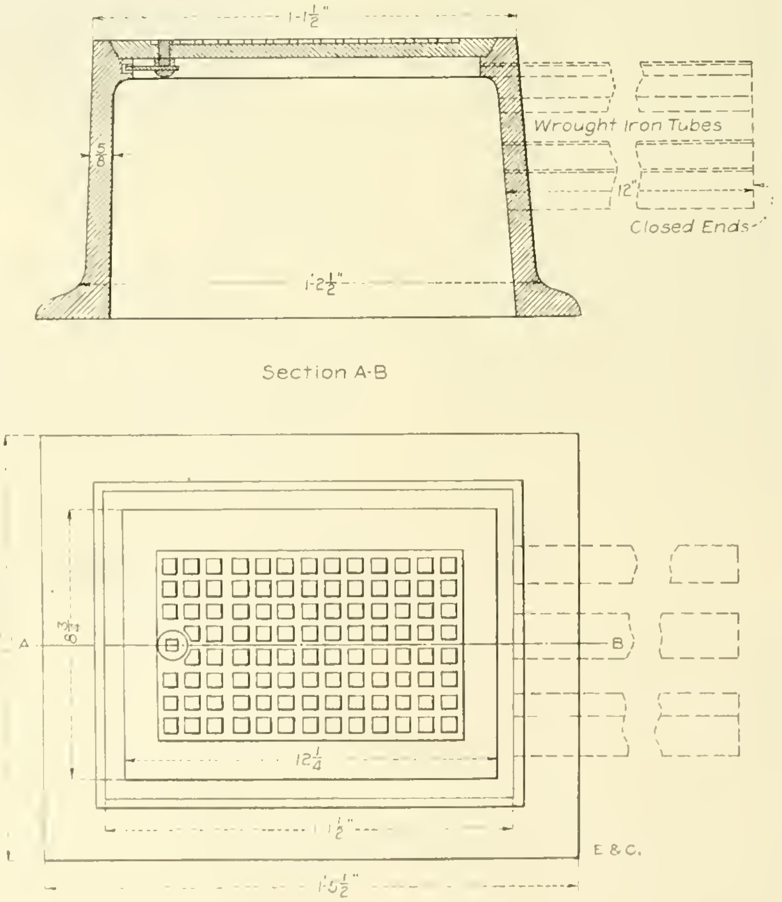


Fig. 2. Thermometer Box to Be Embedded in the Surface of Experimental Roads in Scotland for Obtaining Road Core Temperatures.

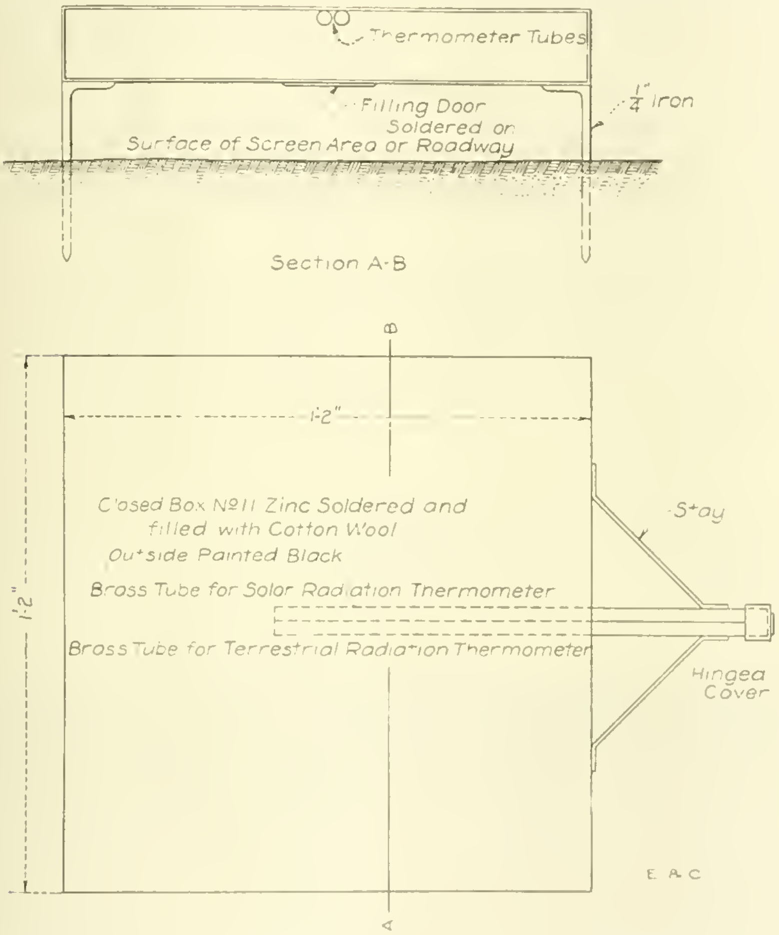
The sunshine recorder proposed to be installed is of the Jordan single cylinder pattern, the principle of which is that the sun penetrates through two small oblong apertures in the cylinder, the rays falling on a chart of sensitized paper contained within the cylinder, leaving a trace resembling a blue print. These instruments are made by Negretti & Zambra, London. The position the instrument is to occupy is shown by Fig. 1.

"Six" brand thermometers, which are combined maximum and minimum thermometers, are to be used to ascertain the maximum and minimum temperatures in the body of the road at various depths. It is suggested that observations should be made at depths of 1, 2 and 3 ins. The thermometers should be placed in the iron tubes, to be set horizontally in the body of the tar-macadam or pitch-grouted road lengths at the depths previously stated, access being obtained to them from the surface box, after described. The maximum reading should be entered under the date previous to that of observation, and the minimum reading under the date of observation. The cost of the surface box and tubes is about 22s. 6d. ($5.47). The arrangement is shown by Fig. 2.

In addition to the records before described, a note should be made each morning of wind, its direction and force, also of fogs, dew, hoar frost, hail, snow, sultry weather and thunder. Columns for these entries are provided on the record sheet.

The instruments named above and also those for measuring wear, etc., are listed by the Committee under the following names and prices:

(1) Attrition gage, etc.—			
At each section:			
3 sockets at 8s 6d each.	£1 5 6		
1 heavy cast-iron cover.	0 6 6		
2 light cast iron covers at 1s 9d each..........	0 3 6		
8 sections at	£1 15 6		
		£14 4 0	
For general use:			
Steel straight-edge and arms ....£	5 0 0		
Davy and Maybury gage	10 0 0		
		15 0 0	
			£29 4 0
(2) Thermometers (Hutchinson's)—			
2 instruments, each 17s 6d....			1 15 0
(3) Viscosity gage (Hutchinson's or similar).........			2 2 0
(4) Meteorological instrum'ts (Negretti & Zambra, London)—			
1 rain gage and measuring glass ....£	1 2 6		
1 Stevenson screen.....	3 10 0		
1 maximum thermometer	1 1 0		
1 minimum thermometer	1 1 0		
1 terrestrial radiation thermometer	1 1 0		
1 solar radiation thermometer	1 1 0		
1 hygrometer	1 15 0		
3 Six's thermometers, each 13s	1 19 0		
1 Sunshine recorder.....	1 7 6		
	£13 18 0		
Less 10 per cent..	1 7 9		
	£12 10 3		[illegible]
(5) 1 metal box (R. and A. Main, Falkirk) for solar and terrestrial radiation thermometers			[illegible]
(6) Cast iron surface box and metal tubes (Watson, Gow & Co., Falkirk), to contain thermometers for road core temperatures ...			[illegible]
			£17 18 [illegible]



Thermometer Tubes
Filling Door Soldered on
1/4" Iron
Surface of Screen Area or Roadway
Section A-B
B
1'-2"
1'-2"
Closed Box No 11 Zinc Soldered and filled with Cotton Wool
Outside Painted Black
Brass Tube for Solar Radiation Thermometer
Brass Tube for Terrestrial Radiation Thermometer
Stay
Hinged Cover
E & C
A

Fig. 3. Mounting of Radiation Thermometers at Experimental Road Meteorological Stations in Scotland.

In running preliminary surveys for the Murfreesboro Electric Railway between Nashville and Murfreesboro, Tenn., recently, the transit party engaged ran eight miles and 3,700 ft. in one day, which is believed to be a record for that kind of work. The party was composed of J. B. Mullins, transitman, a head chainman, and four others.

Standard Street Monuments in Oakland, Calif.

Prior to the placing of the Street Department and City Engineer's office on a salary basis in 1911 there were not more than 30 existing permanent street monuments in Oakland, of which six had been set at the expense of the city. In the course of work connected with a general resurvey of the city by the city engineer's forces, a work which is so extensive that it will require several years for completion, new concrete monuments with cast-iron tops are being placed for marking the street lines. During the fiscal year ending June 30, 1913, 164 of these standard monuments were set, and the department is continuing the work as rapidly as time and finances permit.

The standard monument adopted by the department consists of a concrete shaft 6 ins. in diameter and 3 ft. deep, composed of a mixture of 1 part Portland cement, 2 parts sand and 4 parts crushed stone. The top of the shaft is placed 12 ins. below the street grade, and in the top is set a brass pin ¼ in. in diameter. At about the level of the top of the concrete shaft is set a horizontal ring of concrete, 6 ins. thick, and having an opening in the center 8 ins. in diameter. This opening is lined with a cylindrical iron band which projects about an inch below the concrete ring. A cast iron top having a thickness of ¼ in. of metal, and a 7¼ in. opening is supported by the concrete ring, and is closed with a [illegible] in. cast iron plate as a cover. A sectional view of the monument and cover and the plan of the top are shown by Fig. 1. The castings cost $1.90 per set in 1913.

The last California legislature passed an amendment to the Map Filing Act of 1907, which requires amongst other things, the placing of permanent monuments on all subdivisions of land offered for sale. When the amendment goes into effect the city engineering department of Oakland will insist, if the matter is left to it, upon monuments of the type shown by Fig. 1 being used on all subdivisions within its jurisdiction coming under the provisions of the act.

A Plea for Road Transportation Data.

Contributed by Chester Cooke, Engineer of Roads and Repairs, Jackson County, Mo.

To engineers interested in highway construction and maintenance, it is very gratifying to note the progress being made throughout the country in this branch of the profession; the intelligent, accurate and reliable compilation and publication of cost data relating to different designs and methods is also proving invaluable, but it seems to me that we are relegating the question of "Highway Economics" to a place of minor importance, thereby retarding and retracting the advancement in this branch of engineering. Until the theory and economics of good roads is thoroughly understood by the people their unintelligent and half-hearted co-operation will impede and curtail, to a greater or less extent, the improvement and construction of highways.

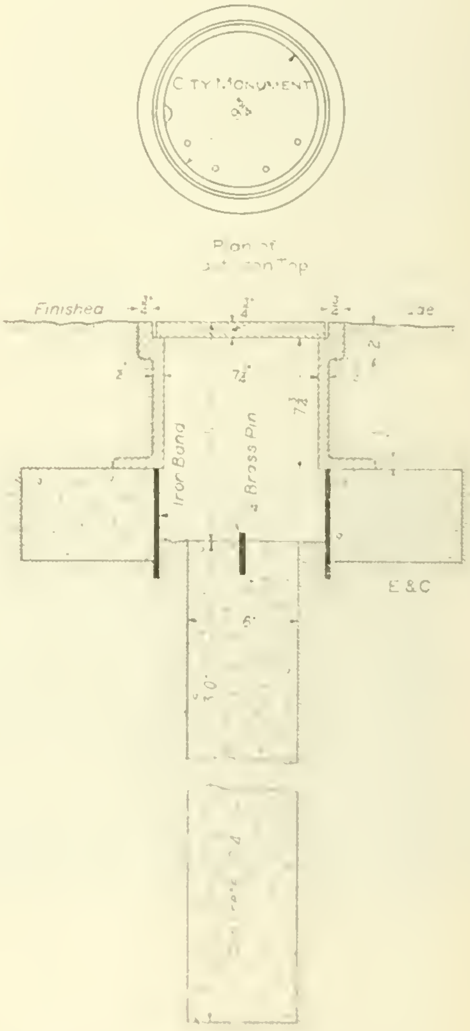


Fig. 1. Section of Standard Street Monument, and Plan of Cast Iron Top, Oakland, Calif.

For instance the majority of people in this country have a general idea of the cost of a mile of macadam road, but if they were to be asked the amount saved to them by this mile in the marketing of their produce or the enhanced value of their land due to this saving the question would be unanswered.

It occurs to the writer, therefore, that a concerted effort ought to be made by engineers in this branch of the profession, to publish any statistics available relating to the cost of transportation on dirt and artificial roads, on maximum and minimum grades, condition of roads and the season of the year when the costs were taken and all other features pertaining to the question. Just how these statistics may best be placed before the public is a detail to be worked out later.

Engineers are, of course, accustomed to analyze and deduce, and to them the importance of this question may not be evident, but is it right that the people should be taxed, asked to vote bonds or appropriate money, as the case may be, with which to construct good roads, when they have only a slight conception as to whether such improvements will pay them a fair return on the investment?

Doesn't it then devolve upon us, as engineers, to show that by the improvement of highways it is possible to reduce the cost of wagon transportation a percentage that would mean an annual saving of millions of dollars?

If such statistics were published more generally the public would soon realize the saving effected by improved highways; we would then have the earnest co-operation of every community and the construction of good roads would not be opposed by injunctions, condemnation proceedings and ill feeling.

An Example of "Road Days" Road Work Near Tarkio, Mo.

Contributed by W. R. Littell, Tarkio, Mo

Governor Major of Missouri, by proclamation, set apart two days in August, 1913, for all male citizens of the State of Missouri to give to work upon the roads of the State. In response to this appeal the County Courts over the State set apart the same days and called upon all public spirited citizens to give two days' work upon the roads of the State in the interests of better roads. The Commercial Clubs and other progressive bodies over the State seconded these steps and took up the details of the work with the citizens in their respective communities. Almost every community had some bad piece of road that had long been an eyesore and a matter of great expense and inconvenience in pulling over it to get to town.

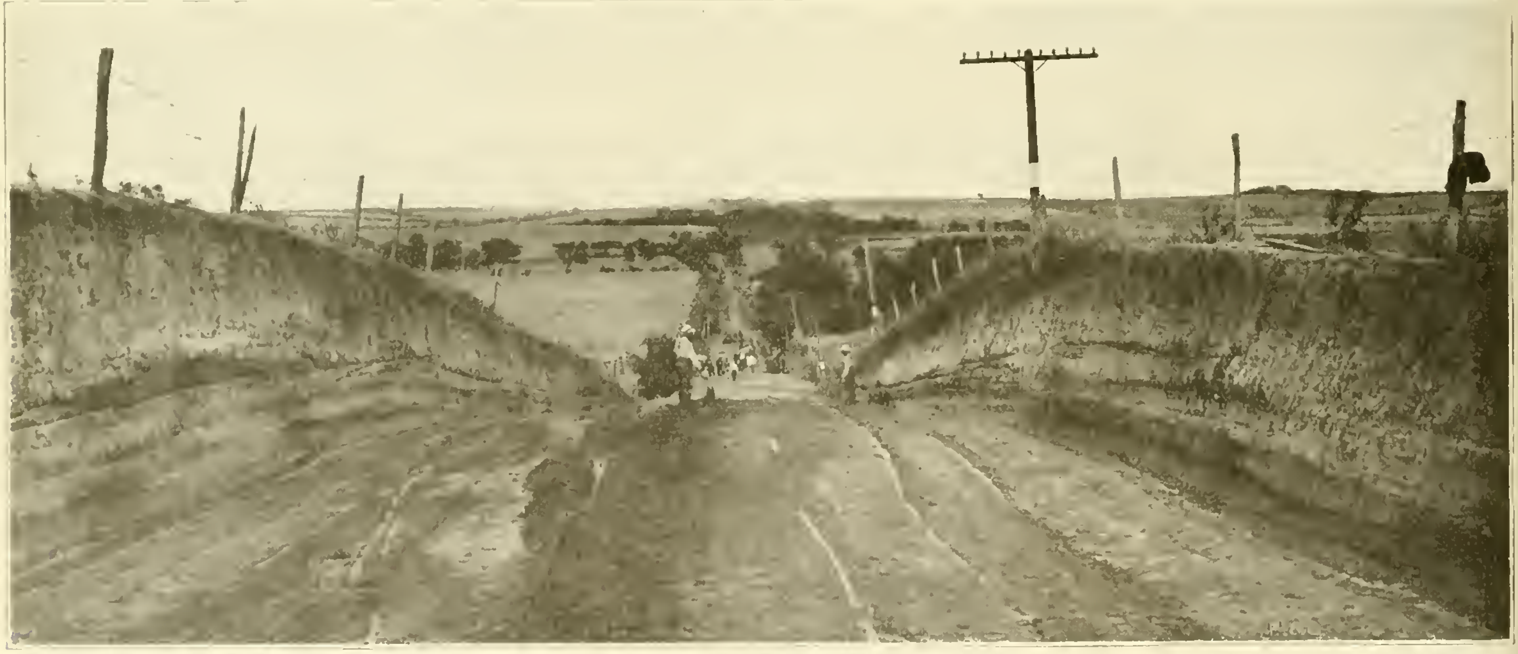


Fig. 1. Gordon Hill 2,000 Cu. Yd. Cut, Near Tarkio, Mo., An Example of State "Road Days" Effort.

These bad pieces of road were singled out and the good roads days were spent on them by the public spirited citizens of the community near them. Such a piece of road was the Gordon Hill, near Tarkio, Mo. It was a hill about 70 rods in length and of about 25 per cent grade before work was begun on it. Just shortly before the good roads days, a man named White lost his life on this hill by an accident in which an auto truck loaded with corn became unmanageable on the steep grades on this hill and ran away, wrecking the truck and killing White.

This hill was marked for the good roads days and the farmers living near it and the citizens of Tarkio and the Tarkio Commercial Club all combined to cut it down to a safe grade. The work was all gratuitous. Thirty teams and 60 men were employed on this work for five days and the hill was cut to a 9 per cent grade. Wagons, plows, spades and wheel scrapers and slips were used. The average haul on the dirt was about 600 ft. Two distinct cuts and two fills were made on the work and a crew worked on each at all times, making practically two crews at work. This road is on the line between two road districts and both road bosses aided in the work, which was all the county contributed.

About 2,000 cu. yds. of earth were moved at a cost of about $850. The soil of this hill is a heavy clayey Maxwell soil and is exceedingly hard to work or handle.

This was the most pretentious piece of road work attempted in the vicinity of Tarkio, although a good many smaller jobs were done. The general plan pursued to get the work out is to call a meeting of all the farmers who have a piece of bad road on which they want work contributed and then secure the names of those who are willing to contribute work and send the number of men asked for to the various jobs on the good roads days. In all, in the year 1913 about $7,000 was contributed in work and money by the citizens of Tarkio and surrounding country, upon the roads leading into Tarkio. In the present year more is expected. We also found in this work that lists of available road tools were convenient, as more men could be had than tools could be found to work with.

The greatest asset, however, to good road work is a lively and aggressive good road sentiment among the people along the road and in the towns at the end of such road. If it is not present good road work does not thrive well.

Method of Constructing a Street Pavement with Flat Laid Vertical Fiber Brick.

Contributed by Hurla W. Kibbey, City Engineer, Grand Island, Neb.

During the fall of 1913 approximately 25,000 sq. yds. of brick pavement were laid in Grand Island, Nebr The pavement was made by laying vertical fiber brick flatwise on a concrete foundation. After such excavation was completed as was necessary to obtain the desired grade and crown, the subgrade was rolled with a 5-ton roller until a solid and compact foundation was secured.

A concrete base 4 ins. thick was laid on the subgrade. It was composed of 1 part Portland cement of standard quality, and 5 parts of Platte River gravel. The gravel contained an average of 3 parts large aggregate and 2 parts sand. It was good, clean gravel, free from quicksand, and the size varied from that of a grain of wheat to that of stones that would just pass through a 1½-in. ring.

A sand cushion 2 ins. in thickness was spread on the concrete base, the required thickness being obtained by means of a template cut according to the crown co-ordinates furnished by the city engineer. Vertical fiber brick 3 ins. deep, 4 ins. wide and 8¼ ins. long, having square edges, were laid flatwise in straight and parallel courses on the sand cushion. At street intersections they were laid herringbone style. In both methods of laying, the brick were laid close together, all lugs being placed in one direction. The brick were subjected to the 1911 National Paving Brick Manufacturers' Association rattler test, and those used were of a class that did not lose more than 15 per cent under that test. After the brick were laid they were rolled with a 4-ton steam roller, and the brick broken by rolling were removed and replaced with sound brick.

Two kinds of joint filler were used, each one being used to cover about one-half of the pavement. Some parts of the surface were covered with Sarco asphalt while the other sections of the pavement were covered with Texico. The asphalt was spread by means of squeegees and sand was then spread on the pavement.

Although the filler and sand were not applied until late in the season—Dec. 19, 1913—and notwithstanding the chilly weather during which much of the spreading was done, it has cohered moderately well. Better results would have been secured had the spreading been done in warmer weather.

This pavement has not had the test of extremely hot summer days and the test of time has not yet proved its merits. The pavement was constructed by the Tynor Construction Co. of Kansas City, Mo., at a contract price of $1.90 per square yard. The same firm will lay about 13,000 sq. yds. of the same kind of pavement in Grand Island in 1914.

The U. S. Reclamation Service's program for the present year calls for the expenditure of over $23,000,000.

Engineering and Contracting

Devoted to the Economics of Civil Engineering Design
and to Methods and Cost of Construction

Volume XLI. CHICAGO, ILL., MARCH 18, 1914. Number 11.

An Instructive Study of Sand-Clay Road Materials.

One of the best of earth roads is the sand-clay road—when it is good. Like most roads constructed of native soils the sand-clay road is in practice of all degrees of goodness. There are some whose surfaces can hardly be surpassed and which under ordinary rural traffic have notable durability and there are others which possess such qualities only in a small degree. To a great extent the reasons which make one sand-clay road good and another sand-clay road inferior are found in the nature and composition of the sand-clay mixture. Presented differently, mixtures of a composition between certain limits will, if foundation and drainage conditions are reasonably good, make a good road, while under the same conditions other mixtures will make an inferior road. These facts are not new nor has it failed to be noted what in general is the range of sand-clay mixtures which give the better results, but the precise analysis of sand-clay mixtures having known service records, to determine wherein their composition lay the property that made one serve well and another less well, is for the first time, we think, recorded in the article published elsewhere in this issue.

The studies referred to are notable because they are not tests of artificial mixtures but are analyses of natural mixtures in service. We have retained of the original paper, though it extends the length considerably, the rather careful directions for conducting sand-clay analyses, but to the practicing road engineer the analysis-history comparison of Tables I and II and the accompanying diagram will with the conclusions be of most immediate interest. Of the conclusions, the first sums up broadly the essential fact to be remembered in selecting sand-clay mixtures. Incidentally it will be noted how closely the findings respecting size and grading of the sand best for sand-clay road mixtures are those which are well known to be true in producing the best hydraulic cement mortars. Indeed one might almost set down as a rule that sand which in size and grading of particles makes the best cement mortar will also make the best sand-clay mixture for roads. By further analogy we may conclude perhaps that the same place of importance which good mixing occupies in the production of cement mortar it also occupies in producing sand-clay road metal.

Soil Formations Underlying Cities.

It is highly important to economic building construction to have at hand data on the character of the underlying soil strata. In some cities these strata are somewhat uniform over comparatively large areas, but in general the subsoil changes considerably in short distances. In Chicago the distance to bedrock is comparatively uniform in most sections of the city, while in New York the rock outcrops in the upper section of the city and lies at a considerable depth in the lower part, often shelving considerably within the limits of a single lot. It is true that engineers often have only a general idea of the character of the subsoil underlying contemplated buildings, and the contractor is generally [illegible] few data upon which to base an estimate of the foundation construction costs. [illegible] possible for him to make borings and test pits, but the time allotted is usually too short for such work. Before deciding definitely upon the type and actual depth of the substructure careful investigations should [illegible] be conducted, but for preliminary estimates and for the use of contractors submitting bids it is to be regretted that more authoritative data on the character of the subsoil in different parts of our principal cities are not available.

As an important contribution to the meager published data on subsoil conditions we wish to commend a paper on "Boston Foundations," by Mr. J. R. Worcester, recently presented before the Boston Society of Civil Engineers, and partially abstracted in this issue. We have given most of the data of general application, but on account of their bulk have not covered the specific data on the character of the subsoil in various parts of Boston and the surrounding territory. These data have been gathered from numerous sources, and the results have been tabulated and condensed in diagrams. The location of each boring is shown on maps, being represented by a numbered dot, and the soil strata are given on separate diagrams. The results of 1,948 borings are recorded in this paper. To facilitate the recording of the combinations of materials found in strata at various depths a certain combination is represented by a number, and in the diagrams this number is recorded instead of stating the materials. The system upon which the classification of the strata is based is interesting and represents much careful work and may well be adopted in subsequent compilations. The materials are grouped into ten headings, each heading stating the material which predominates in the combination. These headings are subdivided into numerous combinations, each subdivision being represented by certain numbers. The principal divisions and the inclusive numbers representing the subdivisions are are follows: Clay or blue clay combinations, 100 to 199; yellow clay combinations, 200 to 299; sand combinations, 300 to 399; gravel combinations, 400 to 499; silt combinations, 500 to 599; mud combinations, 600 to 699; stone or rock combinations, 700 to 799; filling combinations, 800 to 899; hardpan, 900; and wood, 1,000. When the number of combinations rises about 100 for any group the table of combinations then is not in complete numerical sequence, the thousands figure being used only for the purpose of extending the groups which contain more than 100 varieties. For example, 1101 and 2101 represent combinations Nos. 101 and 201 of the "clay or blue clay combinations"; while 1300 and 2300 represent combinations Nos. 100 and 200 of the "sand combinations," etc.

Although numerous test pits and borings have been made in various cities, we find that in many cases complete records of them have not been kept, and in most cases no special effort has been made to classify the different combinations. There is an important field in each of our principal cities for some engineer to collect and record data of a similar character to those reported by Mr. Worcester. It means, of course, a great amount of work but the service rendered to others is noteworthy. We believe that there are engineers in other cities who will volunteer to perform a similar service.

The New Mechanical Filter Plant at St. Louis.

The new mechanical water filtration plant at St. Louis is of immediate interest to all water supply engineers, for the reason that it is the largest plant of its general type in the world. Of more importance than the consideration of great capacity, however, is the fact that St. Louis has installed a mechanical plant after 49 years of recurring agitation for water filtration, and after trying out other processes for the purification of Mississippi River water with only partially satisfactory results. St. Louis was the first American city to give serious consideration to the desirability of filtering its public water supply. Filtration was first recommended in 1865 and the history of the activities of the city looking to the securing of a pure water supply since that date presents many points of interest, chief among which is the evidence of a more lively and appreciative interest on the part of the public in the quality of the city water supply. In this issue we publish an article which states this history in brief, gives a detailed description of the design of the new works and also describes some of the more important features of the construction operations.

The new plant is located within one of the basins formerly used for sedimentation and coagulation at the Chain of Rocks plant, which has been in service for the past ten years. The design and location of the new plant present many features of interest to water works engineers. The new plant will have a rated capacity of 160,000,000 gals. per day, and this output can be increased to 200,000,000 gals. if such a quantity of water must be supplied.

Of special interest to our contractor readers is the fact that five additional contracts on this plant, comprising the head house, the waterproofing of two of the old sedimentation basins, the raising of division walls and the construction of a generator house and a mixing chamber, aggregating a cost of $530,000, will be awarded within the next 60 days.

Reliable Preliminary Estimates of Cost.

Students of the development of engineering practice in this country have often commented upon the inability of the former generations of engineers to make, consistently, reliable estimates of cost of any character. The same observers have not failed to note that in this particular the modern engineer greatly excels his professional forebears. In fact many engineers can now be relied upon to make satisfactory detailed estimates of cost of proposed works, based upon detailed plans, preparatory to asking for contractor's bids. In the matter of making preliminary estimates of cost in advance of the preparation of detailed plans and specifications, however, an estimator is by many considered successful if the actual cost of construction does not exceed his preliminary estimate of cost oftener than one time in four. The difficulty of making reliable preliminary cost estimates is thus emphasized by the frequency with which such estimates when prepared by successful estimators prove inadequate.

The importance of the preliminary estimate is at once apparent when one considers that many projects never get beyond this stage. Such estimates often dictate the decision to go on with a project or to abandon it, and the choice between alternative projects frequently turns upon the preliminary estimate.

Every engineer should endeavor to acquire the ability to make reliable and reasonably accurate preliminary estimates of cost. The process is an art and as such should be studied. It should be recognized as equal in importance to any of the [illegible] aspects of engineering practice.

In this issue we publish an article which sets forth the principles which have been successfully applied in making preliminary cost estimates. This contribution is one previously accepted for publication in our water works section but its appearance in that section should not cause it to be overlooked by readers who are not interested in water supply matters. The illustrations given in exposition of principles relate chiefly to water works but the article is of universal interest to engineers and should be read and studied by every engineer reader of this journal.

The Most Practicable and Effective Measures for Prevention of Damage by Floods.

We publish in this issue a well-written and most interesting discussion of flood prevention and protection. Major Oakes, the author, is a member of the board of army engineers ordered during the period of the distructive floods of the Ohio Valley in 1913 to study these floods and to report upon the subject specified in the title to this editorial. Because he speaks as a member of this board Major Oakes' paper presumably and indeed probably voices in general the conclusions of the board reached in their official report, not yet printed. Engineers have, then, in the article which we publish, a preliminary statement of the board's search and findings. This alone lends exceptional interest to the paper, but we believe that any careful reader will will find the paper of even greater interest as an engineering argument than as a semi-official statement from a board of the Federal Government.

A seriatim review of the arguments presented in Major Oakes' paper would lead merely to repetition; we urge only that these arguments be carefully read and considered. There is one general statement of fact, however, that to us seems to merit special emphasis. It is the responsibility of the people themselves through their own creations for flood damage. This is a truth very seldom remembered, even if it is often realized at all by the general public, and it is a truth that is rarely given due prominence in expert discussion of flood preventation. The fact is presented very forcefully, however, by Major Oakes' discussion so that we requote his statement here as follows:

> The board of which I am a member visited 66 different localities where the greatest amount of damage occurred as a result of last year's flood, and we found at almost every place visited that the damage was largely caused by artificial encroachments on the natural channels. For instance, many towns have grown up on the flood planes of the streams, occupying areas that have been everflowed from the beginning; people have entered the bottom lands and erected their structures with the knowledge that those lands were formed by silt deposited by flood waters; railroads have constructed earthen embankments across these bottoms, leaving only very narrow openings with wholly inadequate capacity for passing floods; city and county officials have built bridges with abutments projecting into the streams, with many piers of insufficient height, thus reducing the discharge area materially, individuals have dumped materials over the banks to increase the area of their property for business purposes. These structures and encroachments have reduced the capacity of the streams, have formed partial dams which raise the water above previous levels and then by the breaking of an embankment or the washing out of a bridge the water held back has rushed downstream under increased head and velocity, destroying everything in its path.

In thinking of these statements care is essential not to confuse the idea advanced with the idea that the works of man have greatly aided in producing floods. It is a debatable question whether any or all of man's works have increased an atom the frequency or the volume of floods. But it is wholly certain that man by his works has largely intensified the damage that floods may work. He has put his structures in the path of the flood, has endeavored to usurp the rights of the water and every year the water somewhere extorts payment for its stolen domain. Major Oakes in bringing out by his paper this one feature of responsibility for flood damage has helped largely to make flood prevention problems more clear.

"Window Dressing" by the Railways.

The Chicago, Milwaukee & St. Paul Ry. is sverely criticised by the Interstate Commerce Commission for acts that financial journals refer to as "window dressing." It is stated that the St. Paul System undertook to make it appear that its 1,400-mile Puget Sound extension show much greater earnings immediately after its completion, three years ago, than actually occurred. Among other things, for example, it is stated that, although the Puget Sound line was opened for regular commercial traffic on Aug. 1, 1909, nearly $500,000 of interest on bonds that accrued after that date was charged to construction. According to the accounting rules of the I. C. C. all interest should have been charged to income instead of to capital account after the construction period had ceased.

Without attempting to pass upon the various criticisms of the railway company, we desire to call attention to a very serious criticism that may be directed against the I. C. C. accounting system. Briefly stated it is this: While provision is made for charging interest during construction to capital or property account, no provision whatever is made for charging the sequel to interest during construction which is now called "development cost." Every civil engineer who knows much about the financial history of railways knows that true net earnings immediately following the construction period are insufficient to pay all the interest on the capital invested in a new railway line. Courts, in passing on rate cases, have frequently recognized this fact, and several railroad commissions, notably the Wisconsin Railroad Commission, have recognized it in appraising public utility properties. *But not a single commission, including the I. C. C., has as yet prescribed a special account to which development cost can be properly charged.*

It will be answered that the railways have themselves not asked permission to open an account to provide for development cost. But why they have not done so is quite evident to any one familiar with present methods of financing. New railway lines are built, for the most part, by old companies that have large surpluses and net earning capacity greatly in excess of outstanding bonds. Under such conditions it has seemed wisest to provide for the deficits of the new lines out of the surplus earnings of the old lines and to defer adequate dividends until the lean years were passed. Thus, intentionally or otherwise, the general public has been somewhat deceived as to the real earning power of new railway extensions, and has been more readily induced to buy securities issued for railway extensions.

In the early days of railway construction large land grants from the government were supposed to suffice to offset the development cost of new lines, but, in fact, such grants seldom did suffice, as many a railway receivership bears witness. The era of land grants passed away, and there came the era of financing new lines with the financial backing of old lines. Then, as above stated, the development cost of the new lines was covered up by the surpluses of the old lines. This covering up was done, in one way or another, even where separate railway companies were organized, and the public rarely knew how long a time elapsed before any new line became able to carry itself.

Within a few years (since 1906) the I. C. C. has had sufficient power and adequate appropriations to study the methods of financing new railway extensions, but it appears not to have done so to any great extent. For if it had made such a study, it would have prescribed a rational method of charging to capital the cost of development; that is, the deficit in net earnings below the interest and dividends on the capitol invested in the new lines.

Even in the appraisal of the railways that the I. C. C. has begun, not a word has yet been given out as to how this development cost is to be valued. That it exists, no one of experience denies. The I. C. C. has issued minute instructions to the railways as to inventorying their physical property, but has given no instructions as to inventorying that very great element of cost—the development cost—which this St. Paul railway incident serves to bring into the limelight. Why, indeed, was the St. Paul Railway so severely criticised and no mention made of the extenuating circumstance that no commission has as yet prescribed an account to which early deficits in interest can be charged?

We are not to be understood as championing the St. Paul Railway, but it seems to us that the I. C. C. itself still lives in a house that has at least one very large glass window

Let us be fair to both sides. The railway company seems to have done wrong in no following the accounting rules laid down by the I. C. C. But the I. C. C. has not done entirely right in prescribing rules that fai to provide a proper account to which the sequel to interest during construction may be charged.

The fact that we are in a transitional period of railway regulation is frequently forgotten by all. New methods of regulation and financing are necessitating some importan changes in accounting, which changes have been only partly worked out. While the I. C. C. property looks to the railways for assistance in solving these problems, it should not look to the railways to take the initiative in departing from precedent. If the old methods of financing and of "window dressing" are reprehensible now, let us remember that they were largely the necessary result of competitive conditions which it is the present object to eliminate.

It is apparently a trait of human natur *slowly* to evolve a new code of ethics and then *suddenly* to apply it, not merely to the present, but to the past. Nothing is mor certain than that even this new code of our will itself become obsolete. And we doub not that some day our present ideas of justice will look fantastic enough when late codes of ethics shall be evolved.

To repeat an apt phrase, "Let us have mor light and less heat" in these discussions o railway financing and accounting. Let u throw the light on the past, if we will, bu never without remembering that we hav traveled far in a very few years, and tha what now seem to have heen wanderings fro a well defined trail were often but groping where no trail then existed.

Suspension Bridge with 451-ft. Span a New Hazelton, B. C.—There has recentl been completed at New Hazelton, B. C., highway suspension bridge with a cente span of 451 ft. and a total length, includin approach spans, of 600 ft. The bridge, whic has a clear width of 9 ft., crosses the Bulkley River gorge, and is about 250 ft. abov the water. The towers supporting the suspension cables are 65 ft. high, and are constructed of fir timbers. One of the tower rests on a concrete pier, while the other i on solid rock. The floor of the bridge consists of 3x4-in. fir plank, laid on 4x7-in stringers, which rest on steel supportin pieces. The cables consist of Cradock's locke coil steel wire rope, 2½ ins. in diamete Each cable has an ultimate strength of 35 tons, and is anchored by means of two 3-in steel bolts into blocks of concrete whic weigh approximately 150 tons. The bridg has steel stiffening girders, designed to carr a moving load of 18,000 lbs. The suspensio rods, which carry the floor system, are ¾ in in diameter. They are spaced 4 ft. apart, an are fastened to the cable with clamps. Th bridge was designed and erected by an English company, all of the steel parts havin been imported from England. The length o time required in actual construction wa about two months. The bridge is in a practically undeveloped section of British Columbia, and is at the site of a primitive woode cantilever structure which was built man years ago by the Indians.

Roads and Streets

An Investigation of Sand-Clay Mixtures for Road Surfaces.

A laboratory and field investigation of sand clay mixtures to determine the characteristics of those giving the best results in road surfaces has been conducted by the Good Roads Department of the University of Arizona. The results of these investigations are given by Mr. John C. Koch in a paper having the above title in Proceedings, American Society of Civil Engineers, Vol. XL, p. 269, and from this paper we abstract and somewhat rearrange the following information:

Sand and clay are the most widely distributed materials in Nature, and, in suitable mixtures, yield a road-surfacing material which can be secured at low cost, because so readily accessible, and when properly built of intelligently selected material, it will render excellent service. Briefly stated, the advantages of sand-clay mixtures for road surfacing are: (1) Low first cost. (2) Low maintenance. (3) No expensive machinery for construction or repairs. (4) No skilled labor required.

Mixtures of sand and clay in proper proportions for road surfacing often occur in Nature, though good results can be secured by making artificial mixtures. The principal advantages in using natural mixtures of the proper proportions, as compared with artificial ones, are: (1) The expense of mixing is eliminated. (2) The natural mixtures become compacted in much less time. (3) Repairs are more easily made, and new materials unite more quickly to the old.

The firmness of the wet sand road and the smoothness of the dry clay road are combined in the sand-clay type, and it remains of almost uniform smoothness throughout the year.

There are many miles of sand-clay roads in Georgia which have cost less than $500 per mile for the surfacing in place. They carry heavy country traffic, and the repairs, during a period of five years have not averaged $5 per mile per year. Considering the fact that the first cost of construction of such roads is less than the annual interest and maintenance charges on almost every other type which has been offered as an improvement on the earth road, it seems inevitable that wherever sand-clay materials are available and of suitable quality, it is the best type to use for the average country road.

Objects of Investigation.—The writer's objects were to determine: (1) The proper proportions of sand and clay in sand-clay mixtures to yield the best results, namely, durability in all weathers and absence of mud and dust; (2) The limits between which the proportions of the two materials could vary without affecting seriously the quality of the resulting road surface; (3) The effect of varying proportions of different sizes of sand in such mixtures; (4) An approximate field method for the examination of soils, to determine their suitability for road surfacing.

Basis of Investigation.—It was determined to base these studies on roads already constructed in various parts of Georgia, and in service from one to five years. In the course of this work the writer has traveled over 2,500 miles of roads, selecting various sections of sand-clay (including "topsoil") roads which had been in use for some time and had worn well, and concerning which definite information could be secured as to materials used, methods of construction, and behavior of the road in all seasons of the year. From the best sections selected, samples of the road surfacing in actual service were chopped out with a hatchet for a depth of 4 ins. in the wheel-tracks. Usually, a 5x5x4-in. sample was taken for laboratory study. Samples were also taken from sections where such roads had not given entire satisfaction, on account of dust and mud. The writer has examined more than 900 samples, representing 48 counties, and comprising every typical sand clay section of the state.

Mixtures of Sand and Clay.—Whether natural or artificial mixtures are used, the properties of both materials which are best adapted to resist the weather and the wearing effect of traffic are utilized, and the unfavorable qualities of both are largely eliminated by using the two in proper combination. It has been shown that clay in the pure state shrinks as much as 10 per cent when dried, and on being wet again will expand to an equal extent. By mixing clay with the proper quantity of sand, these contractions and expansions may be eliminated almost entirely. Therefore, in a sand-clay mixture for road work, the sand, being held together by the clay, which usually more than fills the voids in it, takes the wear of the traffic in dry weather. In wet weather the water quickly drains off this almost impervious mixture, and the sand resists the cutting action of the traffic, although the clay may become puddled to a certain extent.

The writer is aware of the difficulties and intricacies introduced into the subject by the wide variety of clays available and their different characteristics. Nevertheless, he believes that the methods given in this paper, which have thus far proved highly satisfactory in the selection of sand-clay materials in many parts of Georgia, will be of general value wherever such materials are to be found. This basis of study of these materials has been of great service in choosing the best of a number of equally accessible deposits of natural sand-clay mixtures. It may with equal facility be used for determining in just what proportions the different materials should be used in order to get the best mixture possible out of the available local deposits where it is necessary to make artificial mixtures. Most clays contain varying proportions of sand, even when they appear to be almost pure. Clays are often sent in from the southern part of Georgia (where the surface soil is very sandy and it is desirable to lay clay on top of it in order to mix with it and form a sand-clay surface), and on analysis are found to contain as much as 60 per cent of sand.

LABORATORY PROCEDURE.

Equipment.—So much that is useless is often purchased for laboratory equipment that the following list is given for the benefit of those who seek definite information on this subject. These have been found to give quite satisfactory service, and need not cost more than about $50.

One metric balance, 111-gramme capacity, reading to 0.01 gramme.

One metric scale, 2,500-gramme capacity, reading to 5 grammes.

One set of sand sieves, nesting 8 ins. in diameter, Nos. 10, 20, 40, 60, 80 and 100, with dust pan and lid, all brass.

Two steel moulds, brass lined, 1 in. in diameter, 4 ins. long, with 5-in. close-fitting plunger.

One wooden mallet.

Six 300-cu. cm. evaporating dishes, porcelain.

Two Bunsen burners.

Several small tripods for burners.

Several iron wire mats, 4 by 4 ins., for Bunsen flame.

Thermometer, 0° to 100° Cent.

Six 2-quart enameled milk pans for baths, etc.

The evaporating dishes should be Royal Berlin porcelain, as this is the only kind that the writer has found satisfactory.

Separation of Materials.—Samples are usually taken in several parts, so as to enable one to determine to what depth the material may be used. In some cases a sample may be in three or more parts, being 4-in. layers taken in succession from the same hole, each being kept separate from the others. After analyzing each part of a sample separately, the composition of the top 4, 8, 12, or 16 ins. can be determined easily. This gives a clear idea of the value of the material and of the greatest depth to which it may be excavated.

The essential objects of the laboratory examination are to ascertain: (1) Relative proportions of sand and clay, (2) Physical analysis of sand, (3) Physical properties of the mixture of sand and clay, (4) Determination of presence of other material which might prove detrimental, (5) Physical properties of the clay.

To attain these objects, the following steps are necessary, in the order of their importance, and to reduce the laboratory work to a minimum: (1) Separation of sand and clay, (2) Mechanical analysis of the sand content, (3) Slaking test on cylinder of sand-clay, (4) Examination for mica and feldspar, (5) Slaking test on clay cylinder.

Separation of Sand and Clay.—The sample to be examined is first dried thoroughly in the air; then it is screened through a No. 10 sand sieve (10 meshes per linear inch), lumps and clods being pulverized with a wooden mallet beforehand. All coarse materials caught on the No. 10 sieve are considered arbitrarily as gravel, and all smaller materials are taken as the sand-clay portion. Pieces of grass, roots, etc., are discarded, of course, before weighing the two parts of the screened sample. The material held on the No. 10 sieve is regarded as being much the same as the broken stone in a concrete mixture, and the sand-clay as the cement and sand portion constituting the mortar in such a mixture.

The part of the sample passing the No. 10 sieve is then examined as follows: By successive quartering, about 150 grammes are taken and dried in the air bath at 100° C. to constant weight. This may often be dispensed with, and the sample merely air dried. Then 100 grammes of this material are weighed and placed in a porcelain evaporating dish; water is added and the soil rubbed well until the clay particles are in suspension. The clay in suspension is then carefully poured out and more water is added; the process is repeated until there is faint or no coloration of the water when the residue is stirred up. By washing the materials properly, practically all the organic matter, silt, and clay are removed, and only the sand particles are left, with possibly some mica and feldspar.

Mechanical Analysis of the Sand.—The moisture in the residue in the evaporating dish is then evaporated, leaving the sand dry and ready to be screened. After weighing the sand it is screened through a nest of five sieves having 20, 40, 60, 80, and 100 meshes per linear inch. The weights of sand caught on each of these sieves are determined, and a curve is plotted showing the weights as ordinates and the sieve sizes as abscissas, based on 100 per cent for the total weight of the mixture of sand and clay. Thus the weights in grammes show as percentages of the total. Figure 1 was prepared in this way, and shows graphically the sand analyses of a number of samples.

Soil Slaking Test.—A test cylinder, 1 in. in diameter and 3 ins. long, is made of the material passing the No. 10 sieve by wetting it sufficiently to make a very stiff paste and after working it thoroughly together, placing it in a metal mould, using a tight-fitting plunger to compact the material as much as possible by tamping with a mallet. This cylinder should be dried to constant weight in the air bath at 100° C. When entirely cooled it should be immersed completely in a glass jar of water at a temperature of 21° C., and the time noted in which it disintegrates completely. Disintegration is assumed to be complete when the cylinder has broken down until the material is standing approximately at its natural slope of repose.

In sand-clay mixtures which have given satisfactory service, the time to disintegrate completely may vary from 2 minutes to nearly 1 hour; usually, it will be from 5 to 20 minutes. This test will give a fairly good idea of the resistance of any sand-clay mixture to the action of water, and, for purposes of comparison, is made more easily and quickly than that to be described for the slaking of clay alone. The most durable mixtures, in general, are those which take longest time to disintegrate. Mixtures containing a coarse, flaky clay may hold their shape for several days before breaking down completely, but this is not a frequent occurrence.

Tests for Mica and Feldspar.—Mica is easily recognized, and its presence need occasion no alarm unless it exceeds 5 per cent of the total sand-clay sample. This is especially true of the Southern Appalachian region, where mica is often associated with the coarser sands, as in the weathered gneisses of the crystalline areas of that region. In excess of about 5 per cent, mica will cause considerable trouble, acting as a lubricant in wet weather and crushing into a fine powder when dry. It is easily detected with the low power of a compound microscope, a magnification of from 30 to 50 diameters being large enough, in most cases, to make out the characteristic laminated structure.

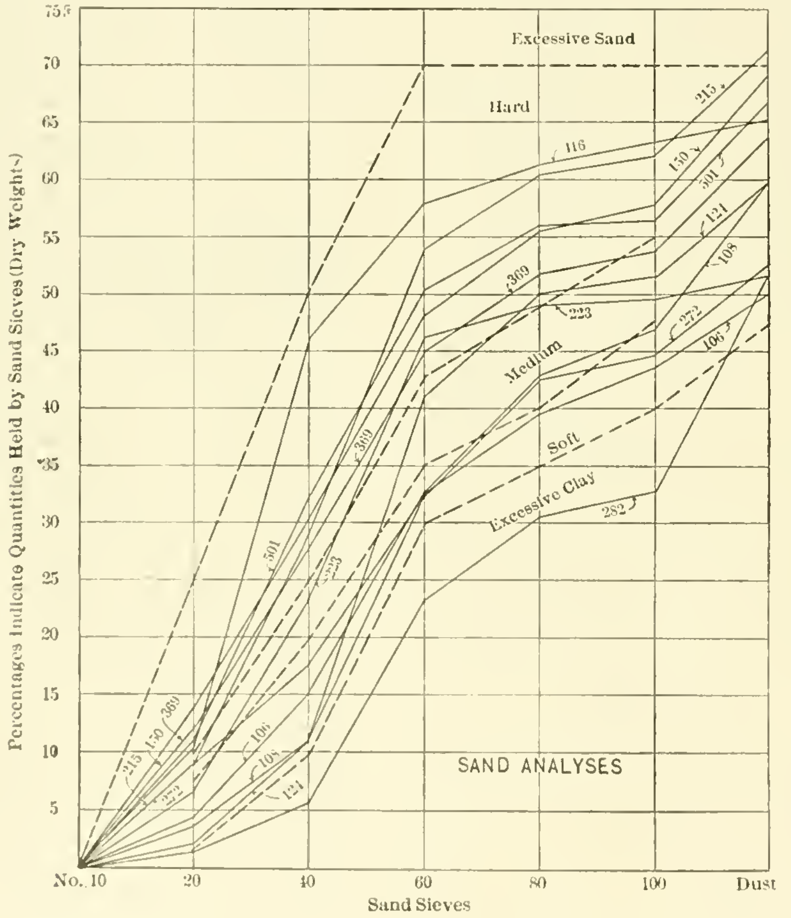


Fig. 1. Graphic Analyses of Sand-Clay Road Mixtures Showing Limiting Curves.

The separation of the mica is often a difficult operation. This is especially true of soils in which it is present in a finely divided state. In washing a sample of such a soil, the mica will go into suspension in the water with the clay and silt. If the clay and silt precipitate before the mica, it is easy to remove the latter by drawing off the wash-water and passing it through a paper filter. Otherwise, it would be impossible to effect a separation of these materials. Soils in which there is much finely-divided mica will usually present a characteristic, lustrous surface.

In most cases, however, the mica can be removed from the sample by washing out the clay and silt carefully (as previously described under "Separation of Sand and Clay"), the residue being sand and mica particles. A separation of these can be readily effected, the difference in the specific gravities being sufficient to permit of a gravity separation when covered with water and gently shaken. The sand sinks to the bottom and the overlying mica can be removed with a spatula. After drying, the mica is weighed and the percentage of the total sample determined.

The most harmful of the feldspathic materials, according to the writer's experience, occurs in the form of small pebbles, or even in large irregular masses, of an earthy appearance, and of a color ranging from light yellow to dark brown or black. Though easily crushed, it will generally have more coherence than the clods of earth or clay in the sample. Therefore, the larger pebbles can be separated by screening the dried sample through the No. 10 sieve. If pebbles of other material are also found, the difference in color will permit of a separation. The feldspathic materials passing the No. 10 sieve, together with the sand, clay and silt, will be left with the sand, as a residue after washing out the other materials. Separation of the feldspar can then be made in the manner described for mica. If this material occurs in greater proportions than about 8 per cent of the total sample, by dry weight, the road surface will cut and wash easily. This is a figure based on actual experience with a few roads, and is probably subject to modification, depending on the characteristics of the other components of any given sample. The presence of fine feldspathic material will be readily apparent in the soil cylinder test. When the soil cylinder is immersed the faldspar will almost immediately go into solution or suspension, and in a minute or so the water will become so discolored that the cylinder cannot be distinguished.

The number of cases in which a separation of either of these materials is advisable will vary considerably. In the writer's experience, about 6 per cent of all samples examined were analyzed quantitatively for mica or feldspar. Although the methods described are approximate, they have given very satisfactory results.

Slaking Tests on Clay Cylinder.—The clay from the 100-gramme sample can be collected by saving the wash-water and allowing the clay to settle. Usually, the quantity of clay from such a small sample is not sufficient to make the test cylinder, so that additional clay must be removed from the material passing the No. 10 sieve to make up the cylinder. This cylinder is made by mixing the dried clay with only sufficient water to make a very stiff paste, which is then moulded under sufficient pressure to make a compact mass. After drying in air and then to constant weight in the air bath at 100° C., the specimen is kept at that temperature until at least an hour after the last trace of moisture is found by condensation on a glass plate above the cylinder. It is important that the cylinder be thoroughly dried. When properly dried it is completely immersed in a glass vessel of proper size containing water at 21° C. The character of the slaking and the time required for complete disintegration depend somewhat on the temperature of the water, and the freedom from moisture of the dried cylinder, so that, for a comparison of results, all specimens should be thoroughly dried and the water for the test brought to the same temperature in each case.

The slaking of clay is similar to that of quicklime, except that the action is entirely physical and not chemical. The time and character of the slaking give an idea of the value of the clay as a binder in the sand-clay mixture. In clays which have been separated from samples taken from road surfaces which have proved satisfactory, the time for the clay cylinder to disintegrate completely varies from 2 to 20 minutes the average being about 4 minutes. The degree of disintegration of the clay may vary from finely divided particles to flakes almost ⅛ in. long. In general, the coarser the particles the better the indication for suitability for road work. Clays which disintegrate completely in less than 2 minutes may be regarded with some suspicion, but need not necessarily be rejected entirely, unless the sand analysis and the soil cylinder also give poor tests.

Typical Examples, with Analyses and Road Histories.—It is thought that a few characteristic cases from actual practice may be of interest. In Table I is given a list of the samples, with the locality from which each was taken.

Figure I shows a graphic analysis of these samples, with the sand analysis in detail. This diagram also gives, graphically, the limits between which the writer classifies his samples, basing such classification on the sand analysis as well as the clay content, both of which are shown.

The limiting curves separating the analyses into four groups are based on the writer's experience, and are given with full confidence in their utility. They are of sufficient accuracy to be of considerable service in classifying sand-clay mixtures properly and quickly, and arriving at a reasonable approximation of their value as road material. Quite often it will be found that the sand analysis alone may show that the material is unsuitable and much time may be saved that might otherwise have been lost in continuing the examination and applying other tests.

The analyses falling in the group marked "Hard" will usually give a durable, hard road surface which wears exceedingly well, and, after consolidation, can be cut only with great difficulty by a road machine, if at all. Analyses in the "Medium" group will give an excellent, smooth, hard surface, but one which may cut a little in protracted wet weather. Such a surface is too hard to attempt to shape up with a road machine, except after very long wet periods. This may be taken as a sort of "average" quality of sand-clay.

The analyses grouped in the "Soft" class give a surface which is much superior to the ordinary earth road, but which will tend to be somewhat dusty in dry weather and will cut easily and tend to wash in wet weather. Material of this type requires reshaping, after every heavy rain, with a road machine or drag. The class "Very soft" comprises all materials which have too large a clay content to give a satisfactory road surface; they should not be used at all, as the expense is scarcely warranted for the kind of road surface that can be obtained with it.

Table II is a summary of the road histories and the corresponding analyses, for compari-

TABLE I.—TYPICAL SAND-CLAY ANALYSES.
(Percentages of Total Weight of Sand and Clay.)

Sample No.—	106	108	124	150	215	223	272	369	416	501	282
Sand No. 10	(12.5)	....	....	(6.3)	....	(24.0)	....	(7.2)	(1.5)	(10.0)	....
Sand No. 20	4.5	3.6	2.0	13.7	9.2	6.8	9.2	12.0	10.6	11.0	1.4
Sand No. 40	10.8	7.6	9.0	16.6	19.5	15.9	8.6	15.7	35.4	21.4	4.3
Sand No. 60	17.5	21.5	30.0	17.7	25.2	23.5	14.5	17.3	11.9	17.8	17.5
Sand No. 80	6.7	10.2	9.5	7.5	6.5	2.8	20.3	6.8	3.3	5.8	7.3
Sand No. 100	4.0	1.0	1.0	2.3	1.6	0.4	2.0	2.0	2.1	0.4	2.1
Sand dust	6.5	13.0	8.5	11.5	9.5	2.1	8.0	10.0	2.0	10.4	19.1
Total sand	50.0	59.9	60.0	69.3	71.5	51.5	52.6	63.8	65.3	66.8	51.7
Clay	50.0	40.1	40.0	30.7	28.5	48.5	47.4	37.2	34.7	33.2	48.3
Sand 20-60	32.8	32.7	41.0	48.0	53.9	46.2	32.3	45.0	57.9	50.2	23.2

Note.—Sand No. 10 is based on gross sample = 100 per cent, and includes all gravel. Remaining portion of table is an analysis of material smaller than No. 10.

106.—Bulloch County.—Had been in service 3 years at time sample was taken. Excellent, hard, smooth surface, but occasionally dusty in prolonged dry seasons. Softens in wet weather and cuts badly then. Re-shaped after heavy rains with light drags. Artificial mixture.

108.—Dougherty County.—Good smooth surface in dry weather. Road in service about 3 years at time sample was taken. Softens and washes in heavy rains. This material, found about 2 ft. below natural surface, is used without addition of other material.

124.—Sumter County.—Very good, smooth surface, but softens and washes badly in wet weather. Re-shaped with light drags after even moderate rains. This is generally true of all the roads in this county, the materials available being very similar in mechanical analyses. Natural mixture occurring 2 ft. below surface.

150.—Clarke County.—Road in use 4 years, with practically none but the very lightest repairs, when sample was taken. Cost of maintenance, as given by superintendent, averaged less than $5 per mile per year. Very little dust, firm, hard road surface throughout the year, though freezing and thawing might at times soften the crust for a depth of an inch or so. No re-shaping with road machine possible, on account of the hardness of the compacted material. This seems to represent the best and most durable type of sand-clay combination yet found in Georgia. Natural mixture found as topsoil of cultivated fields.

215.—Dougherty County.—Road in use about 3 years at time sample was taken. Good hard surface, softens but little in wet weather, and wears well, keeping good shape. Too hard to re-shape by dragging. Few repairs. Natural topsoil.

223.—Richmond County.—Reports of roads on which this material had been used indicate that it produces very satisfactory results. This material is shipped by rail for road construction for distances as great as 100 miles. The analysis in Table I indicates that this is really a low-grade gravel rather than a sand-clay mixture. Yet it is interesting to note that the mechanical analysis of the portion of this material smaller than No. 10 mesh agrees very well with the mechanical analysis of the better classes of sand-clay mixtures which, with very small gravel contents, have given results almost as good. The mechanical analysis is shown on Fig. 1, considering only the portion smaller than No. 10 mesh. This material is dug from a large pit near Augusta, and has been in use for many years.

272.—Habersham County. — The road from which the sample was taken had been in use for about 8 months. The surface is hard and smooth, and softens but little in wet weather. This material was selected by the writer after sampling a large number of local deposits. Haul, about 2,000 ft. Cost in place, for 16-ft. width of surfaced roadway, about $500 per mile. Local officials, after using macadam at about $6,000 per mile, pronounce this a better road than macadam, even if initial cost and maintenance were equal. Local natural mixture of sand-clay from topsoil of cultivated field.

369.—Clarke County.—Built about 3 years at time sample was taken. Hard, durable surface. Few repairs. Too hard to drag. Experience quite similar to that of No. 150. Natural topsoil sand-clay mixture, obtained from nearby cultivated fields.

416.—Hall County.—In use less than a year. Replaced macadam road that cost $6,000 per mile, and gives better satisfaction than the macadam. Cost $350 per mile for 16-ft. width of roadway. Material natural topsoil from nearby cultivated field.

501.—Clarke County.—Road from which sample was taken had been in use about 3 years. This road was surfaced with a very thin layer of natural topsoil from adjoining fields, so that, after being compacted by traffic, its thickness was about 3 ins. In seasons of prolonged wet weather traffic occasionally cuts through and mud-holes are formed. The material is excellent, but has not been used in sufficient quantity to give the best results. A comparison of its mechanical analysis with that of other samples indicates its high quality.

282.—Decatur County.—The road from which this sample was taken had been in use about one year. This is practically no improvement over the ordinary earth road, as the sand content is so low as to be of no value in improving the quality of the road surface. It is very dusty in dry and very muddy in wet weather, after the traffic has been on it a few days. As difficult to maintain as an ordinary earth or clay road. Material used was taken from adjoining fields at a depth of 2 ft.

TABLE II.—SUMMARY OF ROAD HISTORIES AND ANALYSES

Sample No	106	108	124	150	215	223*	272*	369	416*	501	282*
Character of Road Service											
In dry weather	Hard	Hard	Hard	Hard	Hard	Hard	Hard	Hard	Hard	Hard	Medium
In wet weather	Soft	Soft	Soft	Hard	Hard	Hard	Hard	Hard	Hard	Hard	Soft
In prolonged weather	Cuts 2"-6"	Cuts 2"-4"	Cuts 1"-1"	Cuts ½"-1"	Hard	Hard	1"-2" Cuts	½" Cuts	½" Cuts	½" Cuts	6"-10" Cuts
In freezing weather	Cuts 2"-6"	Cuts 2"-4"	Cuts 1"-4"	Cuts ½"-1"	Cuts ½"-1	Cuts ½"-1"	.....	Cuts ½"	Cuts ½"	Cuts ½"	Cuts 6"-10"
Frequency of re-shaping. Times per year	10-20	10-20	15-25	None in 4 yrs.	None in 3 yrs	None in 4 yrs	None in 8 mos	None in 3 yrs	None in 7 mos.	None in 3 yrs.	2-4
Sand, No. 10, percentage	12.5	....	....	6.3	....	31.0		7.2	1.5	10.0	....
Sands, Nos. 20-60, percentage	32.8	32.7	41.0	48.0	53.9	46.2	32.3	45.0	57.9	50.2	23.2
Total sand, percentage	50.0	59.9	60.0	69.3	71.5	51.5	52.6	63.8	65.3	66.8	51.7
Total clay, percentage	50.0	40.1	40.0	36.7	28.5	48.5	47.4	37.2	34.7	33.2	48.3

Notes.—*Data for No. 223, based on road built near Augusta, Ga.
No. 272, Road in use 8 months at time of report.
No. 416, Road in use 7 months at time of report.
No. 282, practically an earth road.

son. For simplicity the portion of the sample larger than No. 10 will be considered apart from that smaller than No. 10.

Effect of No. 10 Sand.—Comparing Sample No. 106 with the others, it is found to contain a larger quantity of No. 10 sand than any of them, with the exception of No. 223. No. 223 is really a low-grade gravel, and is discussed in a succeeding paragraph. The history of No. 106 is not nearly so satisfactory as that of the remaining samples, except Nos. 108 and 282. Sample No. 108 is slightly more satisfactory than No. 106 and No. 282 is practically an earth road. Furthermore, Samples Nos. 124, 215 and 272, which contain no No. 10 sand at all, have given much more satisfactory service than No. 106.

It has been found by the writer, in samples of gravel and chert roads (Augusta gravel being a typical material), that the part larger than No. 10 varied considerably in the few samples examined, although the composition of the part smaller than No. 10 was fairly constant in the proportion of sand to clay. The gravel part of any sample may reasonably be taken as an approximate measure of the durability of a road built with such mixtures. In addition, if a sample has a considerable proportion of gravel and is deficient in sand, the presence of the gravel will largely offset such defect.

From the foregoing it is clear that the effect of No. 10 sand on a mixture may be, either that of making up a deficiency of sand in the portion of the sample smaller than No. 10, or to increase durability when there is no deficiency of sand. Hence, its effect depends largely on the analysis of the portion of the sample smaller than No. 10.

Material Smaller Than No. 10.—Based on their histories, the samples of Table II may be divided into four groups: 1. Ordinary earth, No. 282; 2. Inferior sand-clay, Nos. 106, 108, 124 and 272; 3. Superior sand-clay, Nos. 150, 215, 369, 416, and 501; 4. Low-grade gravel, No. 223.

The principal differences between the two groups are: (1) The total sand content in inferior mixtures averages about 11 per cent less than in superior mixtures; (2) The total quantity of sand from Nos. 20 to 60, in the former averages 15 per cent less than in the latter.

Considering only the second and third groups, the sand-clay analyses may be condensed into Table III.

TABLE III.—COMPARATIVE ANALYSES OF SAND-CLAY GROUPS.

	Inferior Sand Clays. Limits Per cent	Inferior Sand Clays. Average Per cent	Superior Sand-Clays. Limits. Per cent.	Superior Sand-Clays. Average. Per cent.
Sand total	52.6-60.0	56.3	63.8-71.5	67.6
Sand Nos. 20-60	32.3-41.0	36.6	45.0-57.9	51.1
Sand No. 100 and dust	10.0-17.0	13.5	4.1-13.8	8.9
Clay total	40.0-47.4	43.7	28.5-37.0	32.7

Effect of Size of Sand.—Although the quantity of clay is nearly the same in Nos. 106 and 282, the former is a much more durable material. Roads built of the former withstand severe rains far better than those built of the latter material. To account for this, it must be remembered that the analysis of the whole sample of No. 106 is: Sand No. 10, 6.5 per cent; sand No. 20 to dust, 43.7 per cent; clay, 43.8 per cent. The difference in the clay content is only about 4 per cent in the two samples, which seems too little to account for the great difference in their conduct. By reference to the sand analyses of the two samples, it is seen that No. 106 contains nearly three times as much of the No. 20 and No. 40 sands, nearly the same quantities of Nos. 60, 80, and 100 sand and only one-third as much dust, as No. 282. This considerable difference in the ratio of coarse to fine sands explains, in large measure, the difference in the durabilities of the two materials.

CONCLUSIONS.

A study of nearly a thousand analyses, in the laboratory, and a comparison of the results attained with such materials in actual road construction, have led the writer to formulate the following tentative, working rules for examining sand-clay mixtures and as a basis for making up proper combinations for artificial mixtures:

(1) The total relative sand content, disregarding the size of the sand grains, is no criterion of the value of the material.

(2) The sand smaller than No. 60 is of little value in the mixture, that smaller than No. 100, except in very small quantities, is detrimental.

(3) The greater the proportion of coarse to fine sand the harder and more durable will the road surface be.

(4) For the best possible results with sand-clay mixtures, the sand smaller than No. 10 and larger than No. 60 should not be less than 45 per cent nor more than 60 per cent by dry weight of the entire sample. In addition, the sand smaller than No. 10 and larger than No. 60 should be composed of about equal parts

of Nos. 20, 40, and 60. The total sand content should in no case exceed 70 per cent by weight, of the total sample.

(5) Test cylinders of the sand-clay mixture, 1 in. in diameter and 3 ins. long, should, when thoroughly dried in air bath at 100° C., take at least 2 minutes, when immersed in water at 21° C., to crumble down to the natural slope of the material, and preferably should take 6 minutes. If the cylinder fails in this test, it should be regarded with suspicion. If the sand analysis is poor and the cylinder test is also poor, the material is not worth using.

(6) Test cylinders, made from the clay removed from the sample, 1 in. in diameter and 3 ins. long, should take at least 2 minutes to crumble down to the natural slope of the material, when immersed in water at 21° C. If it fails in this test, but passes the test of the preceding paragraph, it may be used, but it indicates a poor quality of binder.

A Study of Thermal and Traffic Effects on Pavements in Kansas City.

Thermal effects on cement filled brick pavements at Cleveland, O., were illustrated and described in our issue of Nov. 27, 1912, in a paper by Mr. James E. Howard, Engineer-Physicist of the Bureau of Standards. The results discussed were obtained from tests conducted by the Bureau of Standards. Later tests have been conducted on Kansas City pavements, and those, with the Cleveland tests, are summarized in a paper by Mr. Howard before the Ohio Engineering Society. We give from this paper the new data from Kansas City as follows:

Experiments at Cleveland and vicinity were begun in the summer of 1911; those at Kansas City were taken in the fall of 1913. At Cleveland the work was chiefly directed toward acquiring data upon thermal effects on grouted brick pavements, in addition to which there were observations on the depression of trolley tracks and the general distribution of wheel loads. At Kansas City tests were made on the effects of changes in temperature on cement pavements, on local effects of wheel loads on different types of pavements, also observations upon changes in slope of an asphalt macadam pavement, the latter a temperature effect.

Referring to the Kansas City work, observations were there made on thermal effects on a concrete paved street, East 31st Street, which had a 50-ft. roadway with concrete 8 ins. thick. At intervals of about 30 ft. contraction joints were provided. Contraction joints, it will be understood, differ from expansion joints in respect to width. They are made by placing sheets of paper against the forms limiting the length of the section and interrupting the continuity of the pavement by means of this separator. Such joints permit contraction at lower temperatures, but put the roadway into a state of compression at higher temperatures.

Diagram Fig. 1 shows certain results which were obtained, results on two gaged lengths being shown, one of which spanned a contraction joint while the other was on the solid concrete slab adjacent thereto. A range in temperature of 35° F. was experienced during the period of these observations. This range refers to the temperature of the surface of the pavement. The measurements were generally made in the early part of the forenoon.

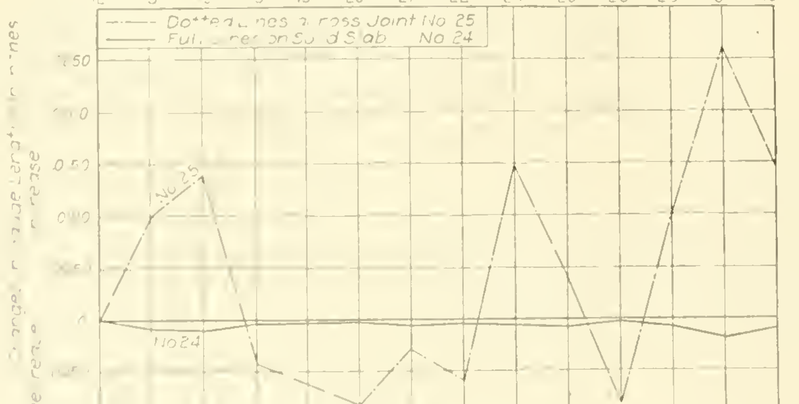


Fig. 1. Diagram Showing Effects of Changes in Temperature on Gaged Lengths of 20 ins. Each, Established on Concrete Roadway 50 ft. Wide and 8 ins. Thick, Contraction Joints at 30-ft. Intervals.

The gaged length on the solid concrete showed a contraction with a drop in temperature as indicated, while the measured results show the extent to which the joint responded and opened in consequence thereof. The maximum opening at the joint observed was 0.0262 in.

Other gaged lengths were established at middle portions of the pavement with reference to the contraction joints, for the purpose of judging the behavior remote from the joints, the gaged lengths being placed both lengthwise and crosswise of the street. So far as now observed, the response to changes in temperature was substantially the same at the middle as at the edges of the slabs. The position selected for these observations was between two embankments, one of which shaded the south side of the roadway, while the other formed an angle on the north side, in which exposure to the sun would cause maximum variations in temperature.

The results plotted on the diagram were obtained on the sunny side of the street. The zig-zag curve representing the movement across the contraction joint indicates clearly the motion taken by the slab as a whole, while the regularity of the curve representing the movement of the solid concrete will also be noted. Surface temperatures do not represent the average condition of the concrete slab and measurements on the solid concrete are not expected to follow in strict accordance with the recorded temperatures.

There were measurements of the effects of wheel loads on different types of pavements, from water-bound macadam to granite blocks. These measurements consisted in measuring the changes in slope of the pavement in immediate proximity to one of the wheels of a loaded truck. For this purpose the truck carried a load of 5,500 lbs. on four wheels. The slope was measured on a chord of 12 ins., showing the depression locally of the wheel load and the resilience was noted when the load was removed.

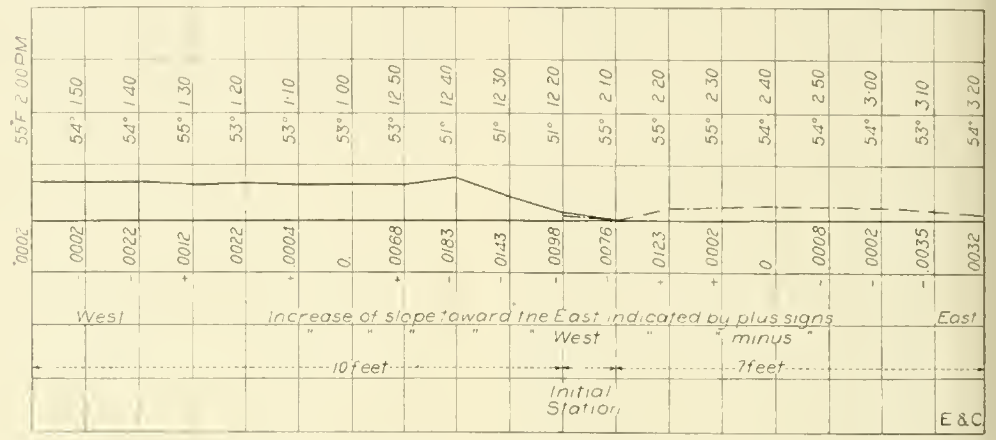


Fig. 2. Diagram Indicating Changes in Slope Along Center Line of Street in Progress During Part of a Day When Surface Temperatures Were 51° to 55° F. Changes Observed in 10-minute Intervals on Chords 12 ins. Thick.

On grouted granite blocks, grouted brick, and on concrete slabs the change in slope ranged from 0.0002 in. to 0.001 in. There was full recovery in the depression upon removal of the wheel load on these surfaces.

On sheet asphalt, rock asphalt and asphaltic concrete the depression ranged from 0.0003 in. to 0.0046 in., the older sheet asphalt displaying the greatest rigidity. Creosoted wood block was rather more elastic and displayed depressions ranging from 0.001 to nearly 0.009 in.

On macadam, there was not infrequently a sluggish action, the full depression not immediately taking place, and upon removal of the load there was corresponding lag in the resilience and more or less permanent set. There were instances in which the material of the roadbed flowed under the wheel pressure and made a ridge which showed a negative depression at the immediate edge of the tire of the wheel, and when such action occurred there was an apparent depression at that place upon removal of the load.

The general behavior of some of the asphaltic macadams indicated that deficiency in binder caused irregularities in depression and that the middle portions of certain roadways were in better condition to sustain traffic than at the edges. The use of a sensitive level in this study of the behavior of pavements and roadways was found very satisfactory.

Diagrams Figs. 2 and 3 refer to observations made on changes in slope of the surface of an asphalt macadam, which occurred during parts of two days. Moderate temperatures prevailed during the hours in which the observations were made.

It was found that a gradual creeping of the surface took place, tending to cause ridges in the pavement, although the temperature during the time did not get above 61° F. The plotted curve on diagram Fig. 2 shows the changes in slope observed during ten-minute intervals. The curve represents, not the profile which would be shown for continuous observations, but from station to station the change which occurred during these short intervals. There were places in the street longitudinally at which the change in slope on chords of 12 ins. showed a rise toward the west and at other places where the slope changed and took the opposite direction. These changes were the result of thermal conditions apparently, since the traffic on the street at the places observed was very light.

Transversely, observations showed an increase in slope toward the center of the roadway, while at the middle of the street the movement oscillated now in one direction and now in the other.

These observations are of interest since they show that creeping may take place in an asphalt surface tending to cause the formation of ridges both longitudinally and transversely at fairly moderate temperatures and without the immediate influence of traffic.

Acknowledgment is made of permission of the Director of the Bureau of Standards to use the advance results on the Kansas City work.

Prizes for Highway Study.—To encourage investigation of methods and materials for road and street construction, and to interest engineering students in highway problems, the Barber Asphalt Paving Co. has offered prizes of $100 for the best paper written by a member of the graduating classes of the leading engineering schools. The title suggested is "Asphaltic Materials for Highway Construction." The paper and its conclusions may be based upon service tests and the lessons of experience; the physical qualities or chemistry of asphalt; or it may combine any two of these lines of investigation. The length of the paper is limited to 3,000 words and all manuscripts must be received not later than June 1, 1914. The purpose of this prize offer is to turn the attention of engineering students to street and road construction as a field of work in which there is great need and great opportunity for trained men.

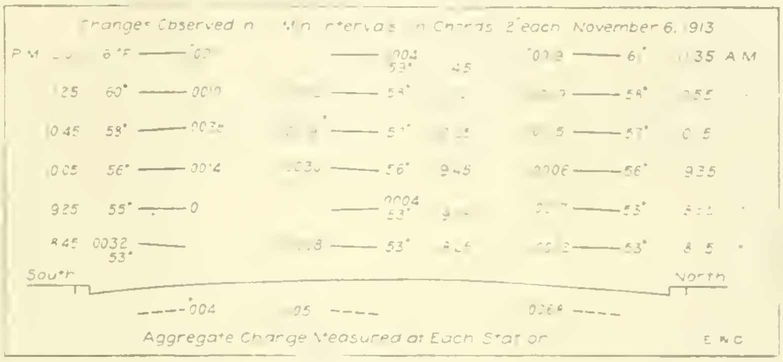

Changes Observed in 10 Min. Intervals on Chords 12 each November 6, 1913
P. M.
A. M.
South
North
Aggregate Change Measured at Each Station
E. N. C.

Fig. 3. Diagram Indicating Changes in Slope Transversely at Center and Sides of Street in Progress During Part of a Day When Surface Temperatures Were 53 to 61° F. Changes Observed in 10-minute Intervals on Chords 12 ins. Thick.

Buildings

Some Features of a Loft Building in Chicago.

Contributed by Harold Doerr, Chicago, Ill.

The seven-story loft building for the Henry Bosch Wall Paper and Paint Co., Chicago, Ill., which was completed in the fall of 1913, covers an inside lot, with a frontage of 50 ft. on Wabash Ave. and a depth of 170 ft. A four-story apartment building adjoins it on one side and a two-story brick building stands a few feet from the property line on the other side.

The rough excavation to a depth of 12 ft. over the entire area of the lot revealed a dry soil made up of a mixture of sand and clay with no trace of gravel. The final excavation and grading removed about 3 ft. of material to allow for the peculiar type of foundation used. The elevation of the bottom of this excavation was almost exactly that of the lake level. This was evident from the fact that no ground water seeped in from the sides of the excavation, and as no quicksand was encountered there could have been no water coming up from below.

The allowable pressure on such soil is given by the Chicago Building Ordinance of 1913 as follows: "If the soil is a mixture of clay and sand, it shall not be loaded to exceed 3,000 lbs. per square foot." This unit pressure on the soil was used in designing the foundation for this building.

The Chicago Building Ordinance also places this type of structure under "Class 1," which calls for a live load on all floors of 100 lbs. per square foot, in addition to the dead load. However, the owners of the building found it advisable to increase this load to 150 lbs. per square foot to fit the conditions which would exist in this building. The columns and foundation were made sufficiently strong to carry three additional stories in case an increase of the business should warrant such an addition to the height of the building.

The construction throughout is of fireproof materials; a steel framework, and terra cotta hollow tile floors, with cinder concrete fill and maple flooring. All elevators and stairwells are enclosed in fireproof shafts. The side and rear brick walls are 21 ins. thick below grade and 12 ins. thick above grade. The facade is constructed of cream colored glazed terra cotta.

FOUNDATION DESIGN.

The unique features of this building are the type of foundation used and the use of old steel columns from another building in the exterior walls. The foundation is of reinforced concrete, covering nearly the entire area of the lot and having a maximum depth of 3 ft. 1 in. (see Fig. 1). The central panels shown in the drawing and marked "open" are the only parts of the lot not covered by the foundation. The column centers are indicated by the "+" mark, and the reinforced panels *A* and *B* are surrounded on four sides by reinforced concrete girders. The cross girders act as cantilevers to distribute the loads from the two central tiers of columns to the soil underneath the reinforced concrete panels *A* and *B*. Table I gives the essential data for each girder.

By noting the plan of the foundation (see Fig. 1) it may be seen that, for all interior columns, the reinforced concrete girders extend in four directions, while for the exterior columns they extend in three directions. The concrete used throughout was a 1:2:4 stone concrete mixture, and the reinforcement consisted of Havemeyer bars of high carbon steel. The girder bars are within 3 ins. of the bottom of the concrete, directly under the column, and are bent up to within 3 ins. of the top surface between columns to resist the negative bending moment in that section of the girder. The two exterior panels are reinforced in both directions with bars placed near the surface for negative moments, hence it is evident that the loads are widely distributed.

The advantages secured by this type of foundation are: First, a saving of time and money; second, greater safety to adjoining buildings, and third, a good distribution of the column loads. However, there were some disadvantages which should be mentioned. The sewer had to be carried along the north wall of the building and given a pitch, so that it occupies some space in the basement. The underground pipe for the sprinkler system also had to run along the basement floor, being put along the south wall. In both cases the foundation made it impossible to allow a trench to be dug in the ground for these pipes.

Before deciding upon the type of foundation to be used for this building three different types were considered: the isolated pier footing, the pile foundation, and the type finally adopted. The isolated pier type of foundation was rejected on account of the fact that it would be necessary to sink wells to hardpan. A pile foundation was designed, but when its cost (although less than for isolated piers) was compared with that of a reinforced concrete spread foundation it was found that the cost was about twice that for the latter type. The cost, based upon the pile foundation, included the required concrete caps for the piles. In addition to the larger cost there was the possibility of damage to the adjoining four-story apartment building. Furthermore, the time required to drive the piles and to put on the concrete capping was estimated to exceed that necessary to build the type of foundation adopted.

A foundation somewhat similar to this one was built to carry the loads of the old Customs House Building of Chicago. In that case, however, there was no reinforcement. A bed of concrete about 5 ft. thick was placed over the entire lot and no attention was paid to differences in the loading, the design being made as though the building was founded on solid rock. That this was not the case became all too evident when the building was completed, the unequal distribution of the loads causing the foundation to crack and to settle unevenly until not a straight sill course or lintel remained on the building.

TABLE I.—NUMBER, SIZE AND SPACING OF RODS IN FOUNDATION GIRDERS

Girder	No. of rods	Size, ins.	Spacing, ins.	Length, ft., ins.	Straight or bent
A	[illegible]	1⅛ round	5½	12-0	Bent
	7	1⅛ round	5½	18-0	Straight
	10	⅞ round	10	7-0	Straight
B	13	1¼ square		25-0	Straight
C	12	1¼ square		27-0	Straight
D	10	1¼ square		25-0	Straight
E	10	1⅛ round	4	30-0	Bent
	10	1⅛ round	4	29-0	Straight
	16	⅞ round	10	7-0	Straight
F	5	1⅛ round	8	26-0	Bent
	[illegible]	1⅛ round	8	18-0	Straight
	12	⅞ round	10	7-0	Straight
G	[illegible]	1⅛ round	4	30-4	Bent
	7	1⅛ round	4	22-0	Straight
H	4	1⅛ round	8	26-0	Bent
	[illegible]	1⅛ round	8	18-0	Straight
I	8	1⅛ round	[illegible]	22-0	Bent
	9	1⅛ round	[illegible]	18-0	Straight
	10	⅞ round	10	7-0	Straight
J	[illegible]	1⅛ round	11	26-0	Bent
	[illegible]	1⅛ round	11	18-0	Straight
	7	⅞ round	10	7-0	Straight

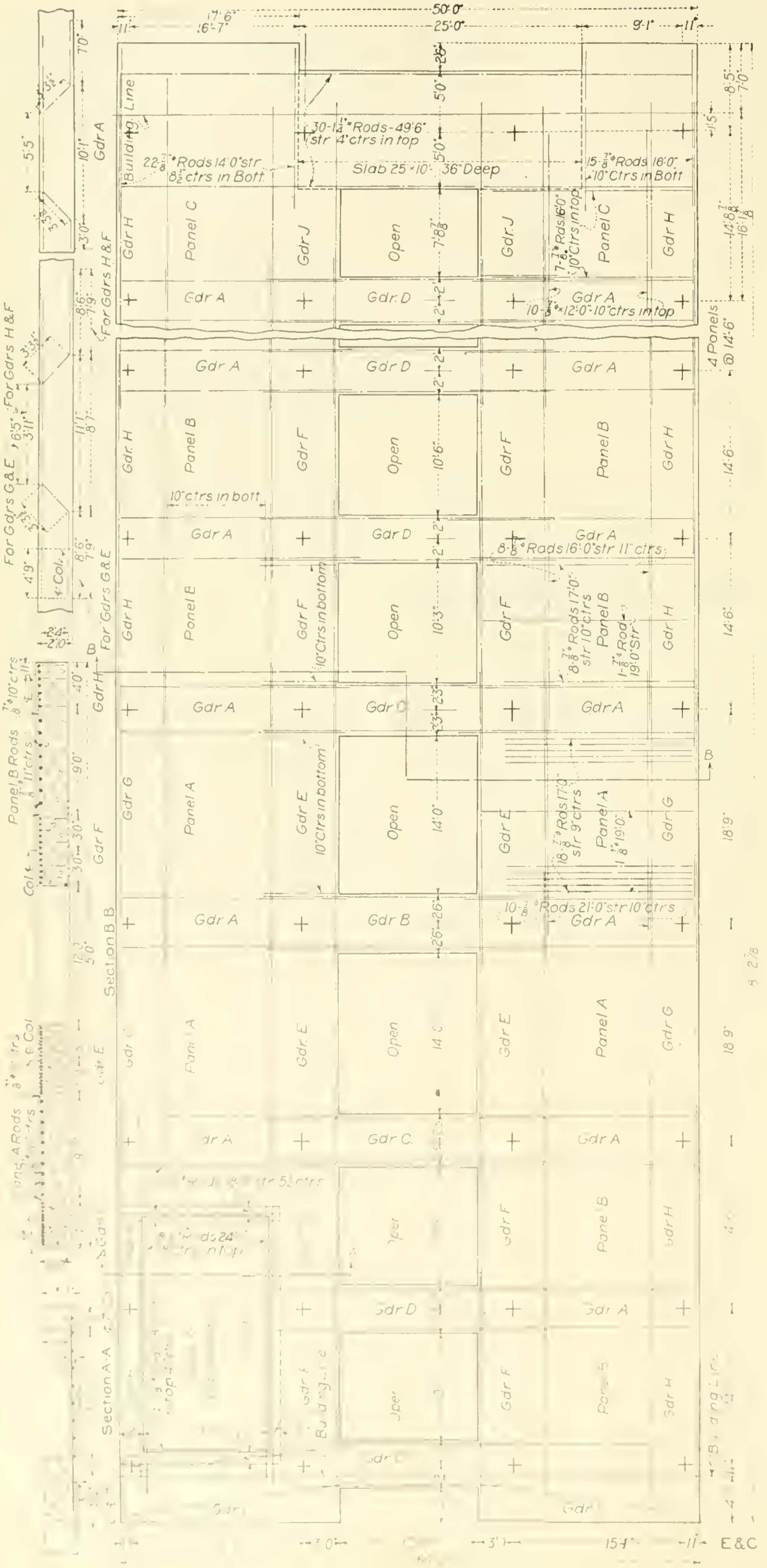


Fig. 1. Foundation Plan and Sections of the Bosch Loft Building, Chicago, Ill., Showing

USE OF OLD STEEL COLUMNS.

Another interesting feature of the Bosch Building is the use of old steel for the exterior columns. These columns came from the old Trude Building which was demolished to make room for the new Marshall Field Building, located at the corner of Randolph St. and Wabash Ave. That this use for steel from buildings which are demolished and replaced by larger structures is a practical solution for its disposal can hardly be disputed. The steelwork in the Trude Building at the time of its demolition, as well as that from other buildings which have recently been torn down, has been found to be free from corrosion or other serious defects. Some damage was done in taking down the steel structure during the cutting of riveted connections, but this was the only cause for injury to the metal. The columns were all scraped and carefully inspected before being repainted and delivered to the site of the new building.

One peculiarity of these old columns deserves mention. At the time that the Trude Building was designed many structural engineers did not deem it advisable to make the lengths between splices for the different columns the same, the contention being that the strength of the building would be greatly weakened at the floors where the spices occurred. In order to overcome this supposed weakness the columns were made of various lengths, no more than one or two adjacent columns having sections of the same length. In adopting these columns to the new Bosch Building some difficulty was experienced in detailing the old plate-and-channel columns, due to these odd lengths. On account of the space required by the old box columns they were used only in the exterior walls. The plate-and-angle columns proved to be much more compact and were therefore used as interior supports where the maximum amount of floor space was required.

As originally designed, all structural steel members of the framework were to be of new material. The steel contractor, however, was given permission to substitute old steel columns from the Trude Building provided they were made as strong as those in the original design. As it was further specified that the material must pass a close inspection, the contract price was not changed on account of this substitution. The exterior columns were designed as plate-and-angle columns, while those actually used were plate-and-channel columns and had a much heavier section than those originally designed. Although this greatly increased the tonnage furnished by the steel contractor, it evidently caused a saving to the contractor and at the same time provided larger sections

EQUIPMENT AND FINISH

As mentioned before, the seven stories now built may later be made part of a ten-story building; hence the foundation and columns are stronger than are actually required to meet the present conditions. The present roof beams, which are to form the framework for a future eighth floor have been set level. The pitch of $\frac{1}{8}$ in. in 1 ft. required for the roof is obtained by means of a graded cinder fill, over which is built a 1-in layer of cement mortar, mixed in the proportions of 1 part cement to 2 parts sand, on top of which was laid a tar-and-gravel roof. There are three pent houses on the roof for the tanks and two elevators in addition to a stair hatch. The building contains a large six-ton high-speed freight elevator, large enough to allow a truck to drive onto it from the alley, and a ten-passenger elevator located in the front part of the building. A sprinkler system of the wet type is installed complete, with pumps for water and air, and a 5,000-gal. pressure tank and a 25,000 gal. gravity tank being provided. The floors throughout the building are of maple, the best quality being used on the first three floors and first-grade factory maple on the upper floors. The 10 in. tile floor arches are covered with a 3-in. cinder concrete fill. All columns are fireproofed with a 1:2:1 mixture of stone concrete, the minimum protection afforded being 2½ ins. All ceilings, stair halls, stair wells and toilet rooms are plastered. The walls of the first three stories only are plastered. Two cast-iron sectional boilers supply the steam for the vacuum heating system.

ERECTION METHODS.

The steel was erected by means of a single-boom derrick, placed in the middle of the building near the north wall. The swinging boom had a length of 105 ft. and a capacity of 40 tons. For raising materials two electric hoists were used, one, located in the front, being used by the masons and later for hoisting the cinder concrete, and the other, located in the rear, for hoisting the terra cotta tiles and for the use of the plasterers. A planked driveway through the central bay of the building permitted materials to be delivered at the hoists and also allowed the empty wagons to drive straight out through the alley at the rear, thus traveling in a continuous direction and avoiding all necessity of backing out. A large vacant lot in the rear served as an ideal place for the storage of the floor tiles, which were all delivered before the first floor was laid.

CONTRACT PRICE AND UNIT COST.

The contract prices of the principal items are:

Steelwork, 703 tons at $60.00	$ 42,180.00
Foundation and sidewalk	11,300.00
Fireproofing	16,000.00
Masonry	14,000.00
Carpenter work	8,500.00
Sprinkler system	7,700.00
Elevators	7,000.00
Heating	6,575.00
Ornamental iron	4,800.00
Plastering	4,500.00
Architectural terra cotta	3,979.70
Cinder concrete	3,900.00
Excavation	2,637.50
Total for above items	$132,872.20

Other contracts increase the total cost of the building to about $117,808. The cubic contents of the building, including foundations and pent houses, are 1,016,965 cu. ft., making the cost approximately 11.5 cts. per cubic foot.

Doerr Brothers, Chicago, were the architects, and L. J. Mench was the contractor for this building.

Design Features of a 220-Ft. Reinforced Concrete Block Chimney at Penarth, Wales.

An unusual type of reinforced concrete chimney has recently been built at Penarth, near Cardiff, Wales, for the new [illegible] plant of the South Wales Portland Cement and Lime Co., Ltd. The following data [illegible] this chimney, were taken from an article by John W. Fisher, in Concrete and Constructional Engineering.

[illegible]

[illegible] it was realized that the chimney would be subjected at times to exceptionally high temperatures, which made it desirable to extend the brick lining to within 12 ft. of the top, and to provide a substantial air space between the concrete and the brickwork as a special protection to the brickwork. Figure 1 (c) shows a detail of one of the reinforced concrete blocks used in the construction of the outer shell; Fig. 1 (d) shows a detail of the top part of the chimney; and Fig. 1 (e) shows a detail at *M*, near the base.

DETAILS OF BLOCKS, LINING AND FOUNDATION.

The concrete of which the blocks of the outer shell are made is composed of materials in the following proportions: 9 cu. ft. of [illegible]

[illegible] the chimney. The vertical reinforcement consists of steel rods fixed in the end joints of the blocks, being further protected by concrete "necks" which are molded as a part of the blocks and which form vertical shafts on the finished chimney (see Fig. 1 c). Each vertical rod projects a distance of 6 ft. into the concrete foundation, and is attached there to a horizontal steel ring which has a diameter equal to that of the chimney at its base. Special reinforcement is used around and over the flue opening and for the molded cornice and necks.

The lining is built throughout of hard, red [illegible] square. [illegible] blocks is [illegible] The total weight of [illegible] is [illegible] uniform load [illegible] tons per square foot on the subsoil.

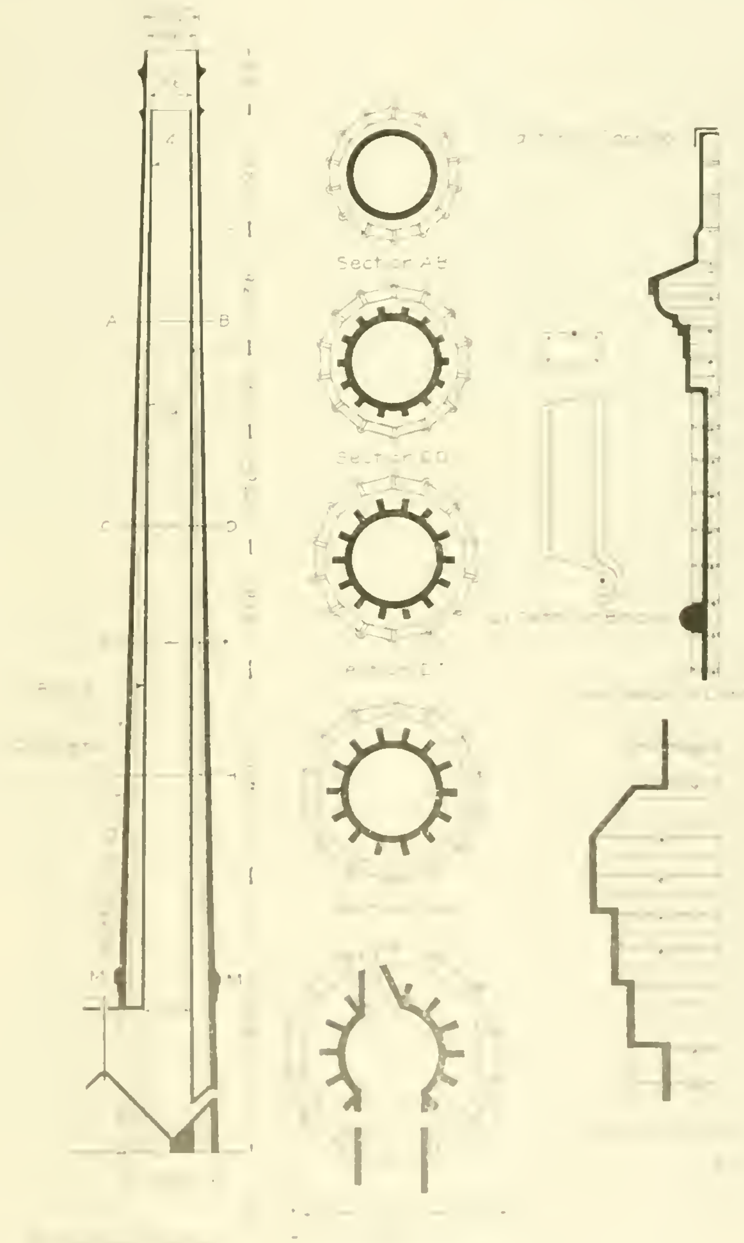


Fig. 1. Sections and Details of a 220 Ft. Chimney at Penarth, Wales

PERSONNEL.

All reinforced concrete work was carried out by Messrs. Monoshaft, under their patented methods. The chimney was built under the direction and supervision of Mr. John W. Rodger, acting as architect for the South Wales Portland Cement and Lime Co., Ltd., associated with Mr. W. J. Cooper, managing director of the company.

Some Data on Boston Foundations and Results of Some Tests to Determine the Bearing Power of Soils and of Piles.

It is highly desirable in constructing the foundations for buildings to have correct data on the character of the subsoil, yet few are available. These data may be obtained by borings or test pits, but in many cases owing to the necessity for rapid construction this part of the work is slighted. The following data were taken from a paper presented before the Boston Society of Civil Engineers by Mr. J. R. Worcester. The original paper contains considerable data on the character of the subsoil at many points in Boston and the surrounding district. In our abstract we have omitted most of the specific data, and have considered those of somewhat general application. The author contributed a paper before the same society on "Boston Foundations," in 1903, and this paper supplements the information given in the previous paper.

GEOLOGICAL FORMATION.

Further experience with the general nature of the formation of the ground in this vicinity has drawn attention to three important conditions not well described in the earlier paper:

First, to the composition of the underlying bed of earth upon which most of our foundations rest. This was described as "clay, free from grit, but containing a large proportion of quartz flour, with occasional thin layers and streaks of very fine sand, and with occasional angular fragments of rock." In many places this description fairly describes the material encountered, but, within the original shore line, there appears to be more of the fine sand formation than would be expected from this statement. In the deep excavations under the Filene Building, at the corner of Washington and Summer Sts., a stratum of clay and gravel from 10 to 20 ft. in thickness was encountered, below which the material was mostly sand of varying degrees of fineness. Most of it was very fine, the grains having a diameter of from 0.003 in. to 0.005 in. When in its natural bed, saturated with water, it had much the appearance of blue clay, showing similar color, hardness and toughness. It would generally stand for some time with a vertical face, even at considerable depth, but in most places it would slowly flow, and in some places it flowed with quite a rush. It would not stay long in suspension in water, but would quickly settle when allowed to stand. When dry, it became light gray in color, and when magnified the particles appeared as nearly clear, round crystals. Under the Jordan Marsh Co. Annex, between Avon Pl. and Bedford St., a similar sand was encountered, but it was much yellower in color. The Cambridge subway tunnel under Beacon Hill was driven through an extremely hard bowlder clay, with occasional thin streaks of sand in its deepest part, but as it approached the surface the material changed to fine sand.

Second, to the consistency of the deep clay formation: This appears to vary in different parts of the Boston basin. Under the Boston peninsula it is generally fairly hard, even near the shore line. In the deep foundations under the Custom House, although fine sand was encountered in places, the greater part of the depth, down to the bowlder clay 33 ft. thick, overlying ledge, that is to about grade —85 (referred to Boston base, which is 0.61 ft. below mean low tide), the material was blue clay, which gradually increased in hardness with the depth. On the other hand, under a section of Cambridgeport and a part of the Back Bay the material is extremely soft, so soft in fact that it apparently is quite free to flow from heavily loaded areas towards places where the load is less. It is not definitely determined, so far as the writer knows, whether such a flowing actually takes place, or whether the clay is gradually being compressed. It is certain, however, that widely-spread settlements have occurred, in some instances to a very marked extent. A section of Cambridgeport covering about 0.5 square mile, centering roughly on Massachusetts Ave. and Albany St., has had a maximum settlement of about 2 ft. (This statement, based on the authority of Mr. L. M. Hastings, city engineer, is at variance with the findings of Prof. W. O. Crosby in a recent report on the land on which the new buildings of the Massachusetts Institute of Technology are being erected.) In Boylston St., between Berkeley and Clarendon Sts., the Transit Commission found the well-preserved remains of a weir or fence at about grade —18. It does not seem possible that this could have been constructed below low-tide level or grade 0. Near Church St. was found a well preserved stump of a tree with roots, at about grade —15. Another instance of subsidence is found in the depth at which peat is encountered. This material must have been formed above water, but is now found, overlaid with silt, far below grade 0. On Tremont St., above Dover, it was found at about grade —12, and on Boylston St. it has been found at grade —19. This tendency to settle will have to be taken into consideration in locating heavy structures in the future. It is not enough to gain the necessary support in piles which may rest in a gravel crust, but the settlement of the crust may seriously injure important structures, as it is believed to have already done in the case of the Public Library and the new Old South Church.

Third, to the danger in mistaking silt for clay: In many places in the city the filling has been put upon a bed of silt from 10 to 20 ft. deep. This silt is very easily mistaken for blue clay. It has nearly the same color, although a little more bluish and not so green in hue. It is tough, nearly impervious to water, soft and smooth to the touch, and when dry is very hard. It usually contains small shells, which are not found in the clay, but otherwise it is dangerously similar. The danger lies in the two facts that the silt is compressible and that it is generally underlaid with peat. Both of these causes render it unsafe for supporting foundations.

GROUND WATER LEVEL.

Another important change which has occurred in the conditions surrounding Boston has been caused by the construction of the Charles River Dam, by which the river above the dam is maintained at a constant level of about grade +8. This would seem to warrant a higher level for cutting piles in those parts of the basin which drain into the Charles River above the dam. The writer is of the opinion that, except where leaky sewers at a lower level drain the ground, or where the ground water is being pumped, it is safe to count on its never standing at a lower level than it does in the basins into which the surplus must drain. This theory is not backed by any comprehensive series of experiments, although the long experience in Boston of cutting piles at the average tide level with entire safety gives some evidence of its soundness. The main argument in its favor, however, is that the rainfall is always in excess of the evaporation, and the surplus water in order to escape must have some head above its outlet. Against this theory it has been argued that in a closely built city so much of the surface is covered with buildings and nearly waterproof paving, only a small proportion of the rainfall can soak into the ground, the most being carried away by storm drains. The answer to this line of reasoning is that though the waterproofed areas keep water out of the ground, they also keep it in by preventing evaporation.

Certain instances have been noted where the ground water level appeared to be far below the outlet, but the writer believes that the cause can be explained without invalidating the theory. One of the most marked cases was when the borings were made in the vicinity of the old Custom House, where it was reported that the water would not rise in the pipes above about grade —30. It was found, however, that when excavations were allowed to fill, the water rose fully to mean tide level. The writer believes that the trouble with the borings was that sufficient time was not allowed for slow seepage through nearly impervious ground. Another case was reported in Brimmer St., where in an open excavation the ground water level for many months was as low as +3, or 5 ft. below the surface in the nearby basin. This case has seemed to the writer to be one of leaky sewers. At any rate, ancient piles recently uncovered in this immediate neighborhood have been found to be well preserved as high as grade +10. In numerous cases the ground water level has been lowered over considerable areas by pumping from deep excavations. This has been very noticeable in Boylston St. and vicinity during the construction of the subway, but here, as elsewhere, it has been found that the normal grade is reached after a time when pumping is suspended. It also sometimes occurs that in an open excavation no trouble is experienced with water until a depth is reached considerably below what might be assumed to be the normal level. This has given the impression that the water level had not been reached, although the cause is simply that the material is impermeable and that, though damp, there is no flowing water. Under such conditions, if time enough is allowed without pumping, water will appear and will rise to the expected grade. In the construction of the Commonwealth Pier, Mr. Hodgdon has adopted +9 as the grade at which to cut buried piles. While this may seem to be rather high, where the mean tide level is +5, it would seem unlikely that near the shore line the ground could ever dry enough at this level to endanger the piles. In view of present knowledge, we may ask whether grade +8 would not be a reasonably safe assumption for low ground water level.

The writer hopes that this point will be fully discussed and that all facts bearing upon the subject will be brought out. It is important not only in determining the safe elevation for cutting piles, but also to determine the possible head that may act on a waterproof floor at low grade.

BASEMENT FLOORS.

The subject of basement floors is perhaps a little apart from that of foundations, though so closely allied that we may be pardoned for going into it briefly. One of the most common mistakes in building construction has been to rest walls and columns on piles, where the soil is soft, and to place the basement floor directly on the ground. The usual reasons for doing this are that it is a great saving in first cost, that it is not expected that the basement will be used extensively, and that it is thought that the weight of the basement floor will be so little that it will not settle even on very soft ground. The results are that in a few years the ground shrinks through the decomposition of organic matter and the floor must necessarily follow. When it is partly attached to walls and columns, as is usually the case, it will crack and drop in the center, forming hopper-like depressions. The writer has seen numerous cases where the settlement has been from 6 ins. to 1 ft. Where the basement floor is below ground water level these settlements are sure to produce leaks; where above water, they may not be so serious, but it is very desirable to have such floors cut free from the pile supports by liberal vertical joints so that the floor may settle in all parts alike. The best way is to give the floor strength enough to sustain itself and its live load, carrying this weight to the piles. If this course is pursued, however, we have a condition where full account must be taken of hydrostatic head acting on the under side of the floor, for a space will soon be left between the floor and the earth which, if the water level is higher, will be filled with water, and it then becomes important to know the maximum head which can occur.

HYDROSTATIC PRESSURE.

There have been many examples of water pressure raising floors, showing that it is an actual not a theoretical condition with which we have to deal. The maximum head has not, so far as the writer knows, been definitely ascertained. In the Back Bay district the streets are at grade 16 or 17, and as the ground water level does not rise to the street grade we have an upper limit and, if the foregoing reasoning is correct, a lower limit between which it is to be located; but they are 8 ft. apart, and this range of uncertainty should be reduced considerably. The writer has assumed grade 12 as the highest water level in many instances, and has not learned of a case where this height has been exceeded on the basin side of the city. Near tide-water, provision must be made for extreme tides, which may reach to about grade 11.

In this connection it may be well to consider how large a proportion of a floor resting on the soil need be assumed to be exposed to water pressure. As has been said above, if the floor is supported on piles, practically the whole area is liable to be so exposed, but when the floor rests directly on the ground, obviously less than the whole area is exposed to water, for part must bear on the soil. Experiments reported by J. C. Meem (Trans. Am. Soc. C. E., Vol. LXX, p. 362) would indicate that with a sandy soil not over 50 per cent is so exposed. The writer has been in the habit of making this assumption in Boston, although realizing that it is purely an assumption, and has yet to hear of its not proving adequate.

CONCRETE.

Since 1903, concrete has almost completely taken the place of stone for foundations, except where exposed to sea-water between high and low tide. Its cheapness and convenience and the ease with which it can be reinforced to resist tensile stresses have given it the place formerly occupied by other forms of masonry and have also nearly driven out of use steel beams and girders in grillages and cantilevers.

Reinforced concrete is wonderfully adaptable to foundations in many ways. Its strength permits the use of thinner walls and slenderer piers. Its density and freedom from joints make waterproofing much easier and more effective. Its application in a plastic condition against a vertical excavation in hard ground gives an easy method of practically filling voids in the earth which, if left open, might cause future settlement of surrounding buildings. Without concrete, the caisson system of foundations would be scarcely possible.

While concrete is an admirable material in every way when it is all right, it has not become so well known that it can safely be used without caution. Unlike rolled steel, it is not manufactured by standard processes in mills doing nothing else, but it is put together on the ground in relatively small quantities and composed of whatever materials can be easily obtained. The selection of materials, especially the sand, is of great importance and requires watchful care at all times. It requires supervision of a higher class than any other common material of construction.

CAISSON FOUNDATIONS.

The use of caissons has not been very extensive as yet in Boston, although they have been adopted in some instances, the foundations for the new Custom House tower being the most notable example. The old building was supported upon large, short, pointed piles which extended only a little way into the clay. The consequence was that on every occasion when the ground water was pumped to a low level in the vicinity the clay contracted and the building settled. At the time the East Boston tunnel was built in State St. it was found that the Custom House was out of level, the southerly side being [illegible] in. lower than the northerly or State St. side. The construction of the tunnel caused a settlement of the northerly side which brought all parts nearly to the same level. The weight of the new tower was considered too great to place on this kind of a foundation, and the architects decided to carry the support down to ledge or the hardpan immediately over it. To do this, it was determined to carry down concrete caissons by means of compressed air. Considerable difficulty was experienced in forcing the caissons down on account of the friction of the soil against the sides, even when the ground was completely excavated under the cutting edge, but it has been successfully accomplished and the caissons filled with concrete.

The sinking of the caissons under the central portion of the building produced further settlements of the original building in spite of the use of compressed air to control the water, but it is thought that no more trouble will be experienced now that the soil has come to rest.

Foundations resembling caissons but constructed in a different way have been used in many instances. Examples of these are in the Filene building. Here cylinders 12 ft. in diameter were sunk under each column. The excavations were sheathed with vertical plank supported by bent steel hoops on the inside. Compressed air was not used and no trouble was experienced with water except in a few places, although a rather large pumping outfit was required when a number of foundations were going down at once.

A similar system has been used by Mr. Charles R. Gow in many places, where he has excavated holes about 3 ft. in diameter within metal plate forms made so that they can be removed as the concrete is filled in. In many of these, where the ground permitted, the bottom of these holes has been enlarged in a conical form to increase the bearing area on the ground. Where the soil does not carry water freely, as, for instance, in silt or peat with the bottom resting on clay, this procedure has been found very efficient and economical.

[illegible]

In the earlier paper certain [illegible] were tentatively suggested for pressures on the soil, but no experimental evidence was offered to support the figures. In the discussion, attention was called to the danger of reasoning from the supporting power of a limited area, on account of the increasing area of the [illegible] of soil below the loaded area, and the fact that with numerous adjacent loads the areas would interfere, thus increasing the [illegible] in each unit [illegible] depth. There is also present in every test a condition having exactly the opposite tendency, which renders them unreliable. The second is, that when a limited area is loaded, the soil has a chance to flow out in every direction and, as the area of the load increases, the opportunity for flow is relatively decreased. The first error, which would be liable to give too high capacities, is important in the case of a harder ground over a softer. The second, which may give too low results, is more likely to be found in the case of a soft plastic material, like clay, immediately under the loaded point.

It has been found difficult to prepare for a test of clay so as to apply the load to soil in its natural condition. The best way that the writer knows is to excavate a pit, large enough to work in, down to a level about 1 ft. above the grade at which the load is to be applied. From the bottom of this excavation a small pit is carefully dug by hand to the desired depth just the size of the plunger at the bottom and with a slight batter to the sides so as not to touch the plunger above the bottom. On the top of the plunger a platform can be constructed to receive the load and this must be held firmly in horizontal position to allow no rocking motion. This can be done satisfactorily by means of boards laid horizontally and nailed to the platform and to stakes driven into the ground at the level of the platform.

TABLE I.—RESULTS OF TESTS TO DETERMINE THE BEARING POWER OF SOIL.

Ref. No.	Location.	Nature of soil	Load, tons per sq. ft.	Settlement, ins.	Time Days.	Time Hours.
1	Kneeland St.	Quite soft clay.	1.74	0.28-0.47	3	
			5.83	0.64-1.21	4	
			6.92	1.62-3.10	6	
			8.02	2.70-4.15	[illegible]	
			9.11	4.30-4.70	6	
2	Harvard St.	Rather soft clay	2.60	0.02		1
			[illegible]	0.08		1
			[illegible]	0.12		1
			6.38	0.19		1
			7.52	0.32		1
			8.65	0.36		1
			9.79	0.44	1	
			10.96	0.59-0.67	1	
			12.12	0.74-1.26	23	
[illegible]	Bennet St.	Hard clay	4.72	0.048-0.1[illegible]	6	
			5.82	0.144-0.18	6	
			6.91	0.18 -0.288	6	
			8.00	0.288-0.36	6	
			9.2[illegible]	0.36 -0.492	6	
			10.18	0.5[illegible] -0.74	10	
[illegible]	Milk and Hawley Sts. (A)	Quite soft clay	[illegible]	0.31	[illegible]	
			[illegible]	0.31-0.38	2	
			[illegible]	[illegible]	2	
	(Load removed and again applied.)		[illegible]	[illegible]	3	
			4.80	1.00	2	
			[illegible]	[illegible]	[illegible]	
5	Milk and Hawley Sts. (B)	Rather soft clay	[illegible]	[illegible]	[illegible]	
			[illegible]	[illegible]	2	
			[illegible]	[illegible]		
	(Load removed and again applied.)		[illegible]	[illegible]	[illegible]	
			[illegible]	0.47	[illegible]	
6	Milk and Hawley Sts. (C)	Rather soft clay	[illegible]	[illegible]	[illegible]	
			[illegible]	[illegible]	[illegible]	
			[illegible]	0.28-0.31	[illegible]	
			[illegible]	0.37	[illegible]	
	(Load removed and again applied.)		[illegible]	0.37-0.47	[illegible]	
			[illegible]	[illegible]	[illegible]	
			[illegible]	[illegible]	[illegible]	
7	Boston & Maine R. R., Charlestown	Medium soft sandy clay	[illegible]	[illegible]	6	

It is to be regretted that there are not more tests on record, for, in spite of the allowances which must be made for misleading conditions, they furnish valuable information. Table I gives the results of the few which are accessible. It is hoped that more may be added in the discussion. The first three were made by the Boston Transit Commission at the bottom of the Washington St. tunnel. The last was a test of a foundation built by Charles R. Gow & Co. for the Boston & Maine R. R.

Except in the case of the last test, these loads were applied to about 1 sq. ft. of soil. The last was a concrete pile 3 ft. in diameter, enlarged at the bottom to [illegible] ft. in diameter. It was 12 ft. in depth. The soil upon which it rested consisted of [illegible] yellow [illegible] gravel. The [illegible] was [illegible] saturated, with [illegible] and in [illegible] was more [illegible]. The soil surrounding the upper portion of the pile was [illegible] for 3 ft. [illegible] clay. The settlement all took place within the first day after the load was applied.

The lesson to be drawn from these tests would seem to be that, even with light loads, clay that appears hard is likely to settle slightly in reaching a bearing, and that for additions to the load temporary increases in settlement may occur. It is probable that where larger areas are covered and the chance of escape of material laterally is reduced, these slight settlements are less noticeable. It appears that with "quite soft clay" it requires from 3¼ to 5 tons per square foot to produce ½ in. of settlement. With "rather soft clay" it requires from 5 to 10 tons to produce the same settlement. With "hard clay" 10 tons has also been found to produce this effect.

These results would seem to show that the recommendations of the former paper, of from 2½ to 4 tons as proper allowances, were conservative, and the same is indicated by the fact that no trouble has been experienced in very many buildings designed on this basis.

PILE TESTS.

A number of pile tests have been observed, the results of which are embodied in Table II. The first experiment in the table was that reported by the late Wm. Parker in his discussion of the 1903 paper; the second was conducted by the late Geo. B. Francis in connection with the construction of the South Station, and the ninth was made by the Sewer Department of the city, under Mr. E. S. Dorr. The last five, made for the Massachusetts Institute of Technology, were contributed by Mr. F. N. Bushnall, vice-president Stone & Webster Engineering Corporation.

TABLE II.—RESULTS OF TESTS TO DETERMINE THE SUPPORTING POWER OF PILES.

Ref. No.	Locality.	Length in ground, ft.	Tip diameter, ins.	Wt. of hammer, lbs.	Fall of hammer, ft.	Penetration, last blow, ins.	Test load, lbs.	Settlement, ins.
1.	B. & A. Warehouse, E. Boston....	40	6	2,050	20	1.92	60,000	0
2.	South Station	27	...	2,000	16	3 to 4	40,000	0
3.	India St.	20 long: 7 in clay	6	1,900	22-23	3	41,740	0
4.	Homeopathic Hospital, E. Concord St.	20	6¾	1,800	15	2	20,000	0
							25,000	⅛
	Note: Loads were applied continuously. During test, another hammer was at work within 30 ft. of the test pile.						30,000	⅛
							35,000	¼
							40,000	5/16
							45,000	7/16
							50,000	⅝
							55,000	⅞
							60,000	1⅜
							65,000	1¾
							70,000	2⅜
							75,000	3 5/16
5.	Same as No. 4..................	31	6	1,800	15	2¾	20,000	0
							25,000	1/16
							30,000	⅛
							35,000	⅛
							40,000	⅛
							50,000	⅛
							60,000	3/16
							70,000	¼
6.	Wentworth Institute, Huntington Ave. and Ruggles St............	37	5½	2,000	20	6	2,000	0
							7,500	1/16
							12,500	⅛
							17,500	3/16
							22,500	5/16
							27,500	⅜
							32,500	½
							42,500	½
							48,000	17/32
							52,500	¾
7.	Sumner St., E. Boston...........	11	7½	2,250	10	2	12,650	0
							26,900	⅛
							35,450	⅜
							44,000	½
8.	Copley-Plaza Hotel, Copley Square	30	6	2,300	17	3	77,000	7/16
9.	Union Park and Albany Sts.....	25	...	2,100	8.6	3½	10,860	1/16
							20,860	3/16
							30,860	¼
	Note: The interval between last two observations was one day.						40,860	5/16
							50,860	⅜
							60,860	11/16
							70,860	1⅞
							70,860	17
10.	Cambridge. Site of new Mass. Inst. of Technology buildings...	28.7	5½	2,300	9	2	7,570	5/32
							12,710	7/32
							22,850	9/32
							28,140	19/64
							33,240	27/61
							48,480	35/64
							53,350	⅝
							58,230	13/16
							61,910	31/32
11.	Same as No. 10....	32.08	6	2,300	10	2	9,280	0
							15,150	1/16
							27,700	⅛
							46,210	5/32
							51,190	7/32
							56,480	¼
							61,690	5/16
							2,300	3/32
12.	Same as No. 10..	41.6	5½	2,300	11	1½	9,020	⅛
							15,290	¼
							20,010	5/16
							30,330	13/32
							56,970	7/16
							62,790	15/32
							68,000	21/32
							2,300	15/32
13.	Same as No. 10....	26.7	6	2,300	10.5	2	7,070	1/32
							18,080	1/16
							28,550	3/32
							34,010	⅛
							51,800	3/16
							58,070	17/64
							62,800	7/32
							2,300	5/32
14	Same as No. 10. ...	23.6	6	2,300	6.3	⅝	11,000	3/64
							14,900	5/64
							20,400	3/32
							25,700	⅛
							30,670	3/16
							35,780	¼
							40,350	9/32
							46,220	13/32
							54,580	7/16
							61,560	17/32
							2,300	3/16

CONCLUSIONS FROM PILE TESTS.

These tests are too few to warrant any very sweeping conclusions, but some interesting deductions may be drawn from them. In each case it will be observed that the ground was comparatively soft, that is, the piles were mainly supported by "suction" or "friction," the end bearing not being much of a factor. In ten instances it appears that small settlements occurred under comparatively light loads and that succeeding increments of loads produced gradually increasing settlements. The writer is inclined to believe that more accurate observations of levels would have detected the same condition in the other tests, but of this we cannot be certain. If we assume that ¼ in. motion is allowable and that more would have been observed and recorded, we could condense Table II into the form shown in Table III.

TABLE III.—DATA CONDENSED FROM TABLE II AND RE-ARRANGED.

Test No.—	P.	W.	h.	p.	$\frac{Wh}{p+1}$.	C.
1	60,000	2,050	20	1.92	14,000	4.3
2	40,000	2,000	16	3.5	7,100	5.6
3	41,740	1,900	22.5	3	10,700	3.9
4	35,000	1,800	15	2	9,000	3.9
5	70,000	1,800	15	2.75	7,200	9.7
6	20,000	2,000	20	6	5,700	3.5
7	30 000	2,250	10	2	7,500	4.0
8	44,000*	2,300	17	3	9,800	4.5
9	30,860	2,100	8.6	3½	4,000	7.7
10	22,850	2,300	9	2	6,900	3.3
11	56,480	2,300	10	2	7,670	7.4
12	15,290	2,300	11	1½	10,120	1.5
13	58,070	2,300	10.5	2	8,050	7.2
14	35,780	2,300	6.3	⅝	8,920	4.0

*Interpolated.

In Table III—

P = the load carried in pounds,
W = weight of hammer in pounds,
h = drop in feet,
p = penetration under last blow in inches, and
C = coefficient which, applied to the formula $\frac{Wh}{p+1}$, will bring the theoretical capacity into agreement with the observed load, that is:

$$P = \frac{CWh}{p+1}.$$

If we disregard test No. 5, which developed unusual capacity, and test No. 12, which settled ¼ in. at a light load, but was afterwards all right, we find that the value of *C* appears to be nearly constant at about 4. The "Engineering News formula," so generally followed, is identical with this, with a value of 2 for *C*, for safe loads. It would appear from these tests that the formula is in satisfactory shape for correlating varying elements, although it does not appear always to have a factor of safety of 6, as it is supposed to have. If we use it with the understanding that it gives a factor of safety of 2 as regards the load which may be expected to cause a settlement of ¼ in., it would seem to correspond quite closely with the experimental knowledge thus far attained.

TABLE IV. — DATA CONSIDERING THAT PILES SUSTAIN THEIR LOADS BY BEARING AND FRICTION.

Ref. No.	P., lbs.	Tip diameter, ins.	Length, ft.	End bearing, lbs.	Friction, lbs.	Area, sq. ft.	Coefficient, lbs. per sq. ft.
1	60,000	6	40	1,980	58,020	104.8	553
2	40,000	6	27	1,980	38,020	61.3	620
3	41,740	6	20	1,980	39,760	41.8	950
4	35,000	6¾	20	2,500	32,500	45.8	710
5	70,000	6	31	1,980	68,020	73.5	930
6	20,000	5½	37	1,660	18,340	89.4	205
7	30,000	7½	11	3,060	26,940	27.7	970
8	14,000	6	33	1,980	42,020	70.7	600
9	30,860	6	25	1,980	28,880	55.7	518
10	22,850	5½	28.7	1,660	21,190	62.8	338
11	56,480	6	32.08	1,980	54,500	77.0	706
12	15,290	5½	41.6	1,660	13,630	104.8	139
13	58,070	6	26.7	1,980	56,090	60.6	915
14	35,780	6	23.6	1,980	35,800	51.5	633

Considering the same piles as sustaining their load by end bearing and by "friction" or "suction," Table IV may be arranged. The area of pile is estimated on the assumption that the taper would increase the diameter 4 ins. in 20 ft. Where the tip diameter is not known it is assumed as 6 ins., and the end bearing is assumed at 5 tons per square foot.

The variation in coefficients is not very great, except in the case of tests No. 6 and No. 12. The reason that the coefficient for test No. 6 is so low is that while the pile was long, the greater part of the length was driven through a silt which adds little to its carrying power. Test No. 12, as explained above, had a rapid penetration at first but brought up later. Disregarding these tests, the average coefficient is 628. If we use a coefficient of 300, we would have about the same factor of safety as with the Engineering News formula and apparently about equally consistent results. It is well to note, however, that it is not always safe to take into account the portion of the pile which is embedded in an inferior material, and the objection to the use of this method is the uncertainty as to how much length to consider.

SUMMARY AND RECOMMENDATIONS.

In conclusion it may be well to summarize the recommendations based on the foregoing data and arguments:

(1) In designing foundations, preliminary borings are of great value to assist in determining: (*a*) At what level, if at all, firm soil may be reached; (*b*) whether piles are likely to be an advantage, and if so, about how long they will probably be; (*c*) in case a hard crust is encountered, its thickness and the nature of the underlying material.

(2) It is of great importance to support all parts of structures on a stratum of soil below the silt and peat.

(3) The maximum and minimum levels of the ground water should be determined. The maximum to obtain the hydrostatic head on waterproofed basements and the minimum to determine a safe level for cutting wooden piles. Subject to review in case the discussion should prove inaccuracy, the writer suggests grade 12.00 (referred to Boston City Base) for the higher and grade 8.00 for the lower limit.

(4) As tentative suggestions for safe bearing pressures on soil, the writer proposes the following:

Material.	Safe load, tons per sq. ft.
Dry, hard, yellow clay, "bowlder clay," dry sand or gravel	6
Compact, damp sand, hard sandy clay, hard blue clay	5
Medium blue clay, whether or not mixed with fine sand	3½
Soft clay, running sand (confined)	2½

(5) For the safe supporting power of piles, the formula

$$P = \frac{3Wh}{p+1}$$ (Engineering News formula + 50 per cent.)

P = supporting power in pounds,
W = weight of hammer in pounds,
h = fall of hammer in feet and
p = penetration in inches.

Water Works

The Art of Making Rapid and Reliable Preliminary Estimates of Cost.

Contributed by Allen Hazen, Consulting Civil Engineer, New York City.

Estimating the cost of constructing proposed or existing structures rapidly and surely is an art. It is a valuable art and deserves cultivation. The success of many undertakings depends upon its use. Some men have the knack of estimating; others can never learn it. But good methods are essential and these can be studied and perfected and applied to special problems.

Estimates made for different purposes, are prepared in quite different ways according to circumstances. They may be conviently classified under three headings:

(1) Preliminary estimates, being estimates made in advance of the preparation of detailed plans and specifications for the purpose of discussing a project for deciding as to its adoption, and for making the necessary financial arrangements for carrying it out.

(2) Detailed estimates, being estimates based upon detailed plans for the execution of the work, and usually made shortly before bids are asked for the particular work covered, or in advance of undertaking to carry it out by day labor.

(3) Final estimates, being the estimates to the contractor at contract prices for the actual work done. The term "final estimate" may also properly be applied to a statement of the cost of a completed work based upon actual expenditures made for carrying it out.

That which follows relates only to preliminary estimates.

Preliminary estimates are made much more frequently than others because only a fraction of the projects for which estimates are made are carried out. Less precision is expected in preliminary estimates than in detailed or final estimates, but on the other hand all reasonably attainable accuracy is desirable, for many important matters depend upon it. The decision as to which of two or more alternate projects is to be adopted frequently turns upon the preliminary estimates of the respective costs. The decision as to whether or not to undertake a certain enterprise usually turns upon the preliminary estimate of the cost of the work. A certain degree of reliability is, therefore, essential in preliminary estimates. On the other hand, preliminary estimates must be made for many enterprises that will never be carried out, and it is necessary that they should be made rapidly and without undue expense to the client.

BASIS FOR PRELIMINARY ESTIMATES.

There is only one real reliable basis for preliminary estimates. It is, the consideration of final estimates of work previously carried out, of a character and magnitude as nearly as may be similar to that of the proposed work. The more nearly the work represented by these final estimates approaches in all the conditions that for which estimates are being made, the more reliable, in general, are the perliminary estimates based upon it; and the more numerous and greater the points of divergence, the less reliable is the basis and the greater is the probable error in the resulting preliminary estimate.

ESTIMATES IN VALUATION PROCEEDINGS.

When a property such as a water works property is to be valued for the purpose of sale, it is common for engineers to make estimates of the cost of reproduction for the structures. These estimates may be in the nature of final estimates when the structures were recently completed and the actual costs are shown by records. They may be in the nature of detailed estimates when full plans and quantity schedules are available. More often they are in the nature of preliminary estimates because such detailed information and cost are not available. Preliminary estimates made in this way are commonly subject to comparison with like estimates made by engineers representing the other party to the transaction, and it is frequently necessary to harmonize such estimates by arbitration or otherwise. If they are presented as evidence in court the engineers who made them must support them through a searching cross-examination. The criticism of preliminary estimates made in this way is likely to be more searching than that of estimates made in the ordinary course of business, where the estimates are made for the purpose of construction and comparison only. For this reason the experience gained in valuing properties for the purpose of sale and in condemnation cases is more useful than any other experience that an engineer can have in training him in the first elements of sound procedure in making preliminary estimates.

METHODS OF COMPARISON.

The problem presented in making preliminary estimates is this: given the final estimates of a certain number of works, more or less comparable to the one proposed, and a general outline of the works for which an estimate is to be made to find the probable fair reasonable cost of construction of the proposed works. The general procedure is to find the elements of cost in the work that is to be carried out, and the actual cost of those elements in the works for which final estimates are available, and to apply the latter to the former. In doing this all known differences that would affect, to an important extent, the probable cost of the work, must be taken into account and allowed for to the best of the estimator's ability.

BASIS OF ESTIMATE.

An estimate may be said to be low, fair, or liberal, according to the methods used in making it. A fair estimate may be defined as one such that with the work carried out in a business-like way under average conditions, there is somewhat more than an even chance that the work could be completed within the estimate. In the case of structures of types that have been built frequently, and of which the elements of cost have been well determined, experience indicates that it is possible in most cases to approximate, in preliminary estimates, the cost of new structures within 10 per cent of the actual cost. With such structures a fair estimate would be somewhere between the most probable cost and 10 per cent more than this, or, in a general way, 5 per cent above the most probable cost. With an estimate made in this way, it should be possible to keep the actual cost of construction within the estimate three times out of four, and this is figuring as closely as an engineer can be expected to do.

In the case of structures of greater novelty, or structures involving undetermined underground conditions, greater fluctuations must be anticipated and the above-mentioned percentages should be increased.

ERRONEOUS METHODS OF REACHING ESTIMATES.

In a valuation proceeding, it is frequently surprising that reputable engineers will present on different sides, estimates differing so much for the same item. These differences must frequently grow out of the use of erroneous methods of procedure by the respective engineers. It may be well, therefore, to consider some of the commonest methods that are erroneous, and to point out the reasons why they should not be used.

The Contract Price Method.—The columns of the technical journals contain a record of the prices at which many contracts are awarded. By going through these columns and selecting the low bids and applying them to the proposed work, an estimate may be prepared which will probably be much below a fair estimate for the work. Among the reasons that it would probably be low are the following:

First, many of the low bidders have underestimated the difficulties of the work, and their bids are really too low. That is, they are not sufficient properly to carry out the work and make a fair profit. The contractor may not be able to complete the work under the contract and additional expense to the owner will have to be met.

Second, in applying the bid prices, or contract prices to new work, it is very easy to overlook entirely some items in the work. A price that relates to a part of the work may be taken as applying to the whole of it through ignorance or failure to make a fair and full comparison.

Third, the schedule of quantities prepared by the engineer for use in the specifications on which bids were obtained may be inaccurate and may not correspond with the final quantities when the work is completed.

Fourth, the items of extra work growing out of conditions that either were not anticipated, or that were intentionally excluded from the contract by the engineer as a matter of policy, are overlooked and ignored.

For these and similar reasons, estimates prepared from contract prices or from low bids reported are almost invariably too low and frequently may be too low by a very large percentage. An estimate made in this way is a low estimate. The figures may be arranged to make a very convincing showing in support of it, but it remains notwithstanding a low estimate.

Estimates Based on Averages.—In many municipal and corporate reports one may find records of the monies actually expended in carrying out certain developments. It is easy to find such records and to compile them and to deduce from them a figure which may be used as representing the probable cost of a proposed work of similar character. Some works represented by such figures may have been done in an efficient and economical manner, and the figures so obtained may be reliable and proper ones for use. Frequently, some or all of the work taken for use in this way was done under conditions that were not efficient, and the cost of doing it may include other items than those relating to the construction work. It is easy to select data in this way to back up a high estimate, and to make a showing that is convincing to those not familiar with the methods.

A FAIR BASIS OF ESTIMATE.

A fair basis of estimate does not exclude either of the above methods, but will take into account data secured under either for what it is worth, and will make allowances for the conditions, or supposed conditions, under which the bids were received, or the work was done. A fair basis of estimate will also give much greater weight to final estimates of work as comparable as may be to the work that is proposed where the work was done under conditions known to be careful and economical. The engineer in making such estimates will naturally and rightly give greatest attention to work done under his own direction, and will give second place to work done by his friends and acquaintances with which he is reasonably familiar as to conditions met and methods used in the construction.

THE WEIGHTED PRICE METHOD.

The number of different kinds of work for which unit prices may be obtained is very great. If an effort is made to keep track of the amount of work of each kind, and to estimate the amount of it in the proposed work, the schedule will be too complicated and the labor of applying the figures will be unduly increased. Moreover, if the schedule used for comparison is too complicated, some items are sure to be overlooked, with the result that too low a final figure will be reached. In order to prevent both of these conditions it is frequently best, in preparing preliminary estimates, to take only a limited number of the main items of work that are involved in the kind of construction that is contemplated, and to weight the unit prices for them in such a way that they will include the whole cost of all the incidental items naturally associated with them.

In proceeding in this way one selects first the items that are to be used. These items should be so selected as to include directly the major part of the work. The final estimates that are to be used as a basis are then taken for analysis. The whole cost of the work represented by each is then distributed among the selected items. Considerable judgment is required in the distribution, and each part of the construction should be included with the item to which it is most nearly related. The most important point is that every dollar spent should be charged under some one of the selected headings.

The sum of the costs reached in this way, divided by the quantities in the final estimate, gives new unit prices which are weighted to include all the minor items. Applying these unit prices to the main items of the proposed work gives a basis for preliminary estimate.

As illustrations of these methods, take the case of pipe-laying. The trenching, the lead, the cast iron pipe, the teaming and incidental expenses may all be represented by separate items in the final estimate that serves as a basis. These can obviously be consolidated into a single item per lineal foot of pipe of a given size. In this process the price for pipe is weighted to include the other expenses that naturally go with it. The process may be carried further and the pipe still further loaded to include the gates, the hydrants and all auxiliary structures.

In a similar way the cost of the reinforcing and of the forms for concrete construction first stated as separate items, may be consolidated, and all the different classes of concrete may be brought into one so that a unit price for masonry includes reinforcing and forms, and all the appurtenances that go with the masonry structures.

In estimating the cost of sand filters the writer has for years divided the whole cost of the construction into four parts, as follows:

(1) Excavation and earth work.
(2) Masonry.
(3) Filtering materials.
(4) Piping and auxiliaries.

The cost of each plant built worked out in this way on a uniform basis affords a basis for rapid and accurate comparisons, and allows the data to be applied in making preliminary estimates for new work where the prime conditions of construction are known with comparatively little chance of large error.

Weighted unit prices of this kind must be carefully obtained and can only be used with caution. The amount of weighting must always be kept clearly in mind by those who use them. When properly deduced and used, they afford an extremely useful and rapid method of approximating the cost of many structures.

THE RATIO METHOD.

There are many cases where the system of weighted unit prices cannot be used because the schedules in the final estimates that serve as a base are in such form that unit prices cannot be deduced from them. For example, there are many cases where the amount and character of work are known and the total cost of the work is known, but there is no way of sub-dividing it between the different items. To compare the costs of different pieces of work with each other, and to get a basis for estimating the probable cost of other similar work is then much more difficult.

A method of reaching an approximate and useful solution of this problem is one which may be called the ratio method. A schedule is made of a limited number of items of work which represent the greater part of the construction in the several cases. A simple schedule of unit prices is then formed corresponding to the units. One fixed price is assumed for each kind of work. The price assumed should be a reasonable one, and as nearly as is known an average one, but precision is not to be expected and a round figure may always be used. The amount of work under each item in each job is ascertained and the assumed prices are applied to them. The sum of the amounts for each job represents what that job would have cost at the assumed prices. A comparison between the actual cost and the cost at the assumed prices gives an idea of the relative economy of the work. It may be found, for example, that one piece of work cost 20 per cent more than the amount obtained by applying the assumed prices; another piece of work cost 12 per cent more and a third, 7 per cent less. When the records of a number of known pieces of work are compared in this way, it furnishes a basis for making a preliminary estimate for work of the same class. In doing this the quantities for the proposed work are ascertained, the base prices are applied to them and a ratio by which the sum so reached is to be increased or diminished is ascertained by consideration of the ratios actually found to have been obtained in the jobs for which cost records are at hand.

In arriving at the ratio to be used the engineer will compare, perhaps in his mind and without written schedules, the ease or difficulty of the proposed work as compared with the ease or difficulty of the various works which served as a base; will take into account differences in labor conditions, in freights and deliveries; will take into account the known or assumed efficiency or lack of efficiency in the execution of the several pieces of work from which his basic data were derived, and will reach an estimate of the addition or subtraction to be made to or from the base price in each case.

This method is commonly combined with the preceding. That is to say, the base prices are usually loaded prices.

This method affords a convenient and efficient method of comparing the relative costs of different works where the loaded unit price method cannot be applied and in experienced hands it affords a rapid and reliable method of making preliminary estimates upon many classes of structures.

EXTRA COST OF NOVEL DESIGNS.

Work on novel designs commonly costs more than work following standard designs. This is true even when well tried methods are first introduced in new places. Such work may be too small to attract bidders from a distance. The unit cost will then overrun anticipated prices. An under estimate of cost is frequently made on work because the estimator fails to realize what a great effect familiarity with the methods of performing work has upon the cost. To realize this one has but to think of the difference between present methods and costs of building tunnels and subways and deep foundations, and the methods and costs of only 15 or 20 years ago. Not only is the risk which the contractor takes now less, but methods which have been thoroughly tried out are at his disposal as well as experienced foremen and laborers to do the work more economically. For a structure of new or novel design much caution in using prices that may be standard on other kinds of work must be used.

SMALL JOBS COST MORE IN PROPORTION THAN LARGE ONES.

Another common cause of under estimating costs is the use of figures on large pieces of work for estimating small work. The engineer often overlooks the large cost of overhead charges, the waste of labor and the cost of plant caused by organizing a force to perform a small piece of work. He too often forgets that for small work the work must be done by less efficient methods or the cost of plant prohibits the use of expensive machines. Perhaps the most common case of such underestimating is where the job, although a large one in the aggregate, consists of many small pieces of work of widely varied character. Such a job is troublesome and costly for the contractor and the experienced man knows it and puts in a corresponding bid.

It must be remembered that bids follow the law of supply and demand; that general slackness in construction work calls out bidders and low prices. The condition of the money market, the cost of materials and the general opinion of the condition of contract work are of course matters of much importance. It seems scarcely necessary to say that the engineer should know where and how the contractor is to obtain his materials and have a fairly definite idea of the cost either in dollars and cents or as a comparative figure to other work.

The engineer in estimating should try to look at the work from the standpoint of the contractor, should try to remember that no work runs as smoothly along as the contractor wishes, that labor conditions and other matters often spoil the best laid plans. He should endeavor to keep in mind the various work he has watched or performed and the numerous times when unexpected conditions added largely to the cost.

Generally speaking, estimates on proposed work by men of limited experience are too low, yet it is not unusual for an engineer to be so impressed with the difficulty of a piece of work that he overestimates the actual cost and it is more common to find that he has overestimated the bids of the contractors.

It is of much value to have two methods of arriving at estimates. If an independent check method can be made even though a rough one, a failure of the two results to agree often leads to the discovery of serious errors in the application of one or the other method.

CONDITIONS OF SUCCESS IN ESTIMATING.

One of the first requisites for successful estimating is a fair and unprejudiced, and moderately pessimistic mind. The estimator must be alert for new and cheaper processes and methods, and conservatively sceptical concerning their merits. Moreover a successful estimator must have had experience in actual construction. As a general rule an engineer should not make an estimate for a structure that he would not know how to build.

Next, as a requisite to successful estimating, may be mentioned a broad basis of cost records of actual work, more or less similar to that for which estimates are to be made. A good and safe method of using the cost data and adjusting it for application to new conditions is equally important.

And, finally, the estimator must have the will to refuse to make estimates for work that he does not understand.

Design and Construction of the New 160,000,000 Gallon Mechanical Water Filtration Plant at St. Louis, Mo., the Largest in the World.

In his paper before the recent annual convention of the Illinois Water Supply Association, Mr. Edward E. Wall, Water Commissioner of St. Louis, Mo., traced briefly the interesting history of water purification in his city, and described the design and construction of the mechanical filtration plant now under construction at the Chain of Rocks. The paper is here given in full, owing to the great importance of the works to which it relates.

St. Louis was probably the first city on the Western Continent seriously to contemplate filtration as a method of purifying its water supply. In 1865 Mr. James P. Kirkwood, then chief engineer of the water works, was sent to Europe with instructions to investigate and study the processes of clarifying river waters there, so that some similar method might be adapted to the Mississippi River water at St. Louis. Mr. Kirkwood reported to the Board of Water Commissioners the results of his investigations in 1865, and recommended slow sand filters, outlining a tentative design for their construction.

The following year the work of building new water works for the city was started at Bissell's Point, although Mr. Kirkwood does not seem to have altogether lost hope for the adoption of his filter project until 1869, when he writes: "The public mind of St. Louis, so far as it has been expressed, does not yet seem inclined to consider filtration important."

The new works completed in 1871, were designed to provide for clarification by sedimentation only, four settling basins being built, each 277 ft. wide by 600 ft. long, and holding about 20,000,000 gals. each. The city at this time was using about 14,000,000 gals. of water per day, so that the basin capacity was more than five times the average daily consumption.

In 1875, when the average daily consumption was slightly above 20,000,000 gals., an increase of settling capacity was deemed an immediate necessity. Two years later, under the new charter of the city, the water works were placed in charge of a water commissioner, Thomas J. Whitman, who at once called attention to the inadequate settling capacity, recommending the passage of legislation to reduce the waste of water, which he estimated at 50 per cent of the total pumpage. Year after year Mr. Whitman emphasized the necessity of enforcing ordinances prohibiting the waste of water, but found the people unconvinced by his logic, just as succeeding water commissioners in later years have met with the same unreasonable opposition on the same subject. So that against his better judgment he was forced to build more pumps to keep up with the consumption. No more settling basins were built and in 1881, during the extremes of temperature the water supplied the consumer was practically pumped directly from the river, since it passed through the basins so rapidly that little or no settlement took place. These conditions continued, increasing year by year in length and frequency of period until, in 1887, it was decided to locate new works at the Chain of Rocks.

In 1885 the Board of Public Improvements recommended building new works at the Chain of Rocks, and the construction of filter beds in the existing basins at Bissell's Point, with an alternate proposition of building the

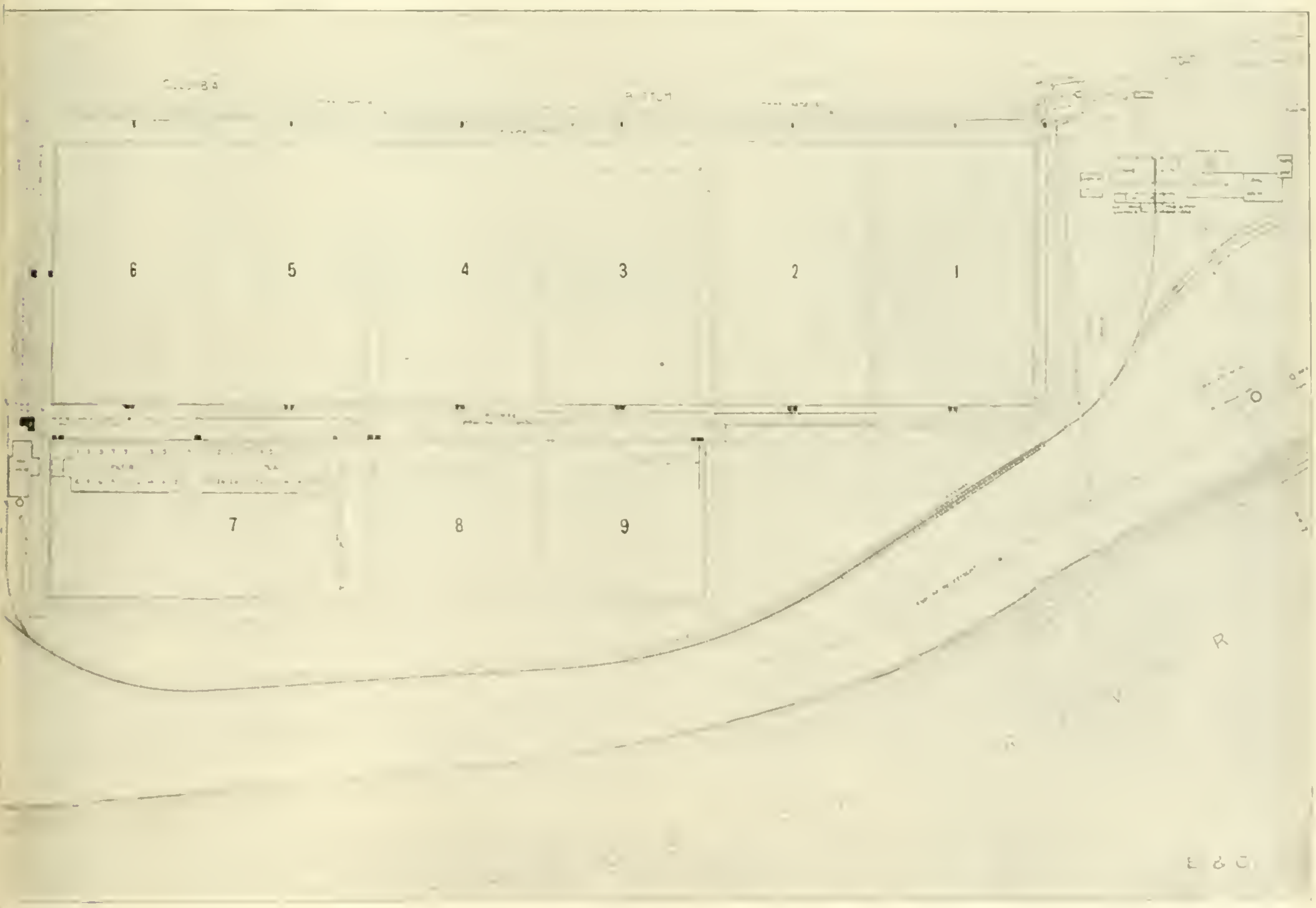


Fig. 1. General Plan of Old Coagulation and Sedimentation Basins of the St. Louis Water Works Showing Location of New Mechanical Filter Plant in Basin No. 7.

filters at the Chain of Rocks at an increased cost of $350,000. The filters proposed at that time were of the slow sand type, estimated to filter 2,000,000 gals. per day per acre, and of sufficient size to supply 50,000,000 gals. per day.

From Aug. 18, 1885, until Feb. 23, 1887, tests were run on experimental filters to determine the comparative value of the upward and downward systems of filtration as applied to the Mississippi River water at St. Louis. The raw water was applied to the filters without the use of any coagulant.

The data obtained in these experiments do not seem to have encouraged the authorities to undertake filtering the river water, so that the construction of the Chain of Rocks low service pumping station was designed with six settling basins, each 670 ft. by 400 ft. in plan, and with a combined capacity of 180,000,000 gals. This plant was put in service in 1894.

Again in 1900 plans for mechanical filters with a rated capacity of 120,000,000 gals. per day were prepared, but the administration was not favorable to the scheme, principally because the people of the state of Missouri, as well as of the city of St. Louis, had been prejudiced against the use of alum in baking powders, and the proposed use of alum as a coagulant in the preliminary treatment of the water supply was sufficient to kill the project. Thus filter construction in St. Louis was defeated for the third time in 35 years.

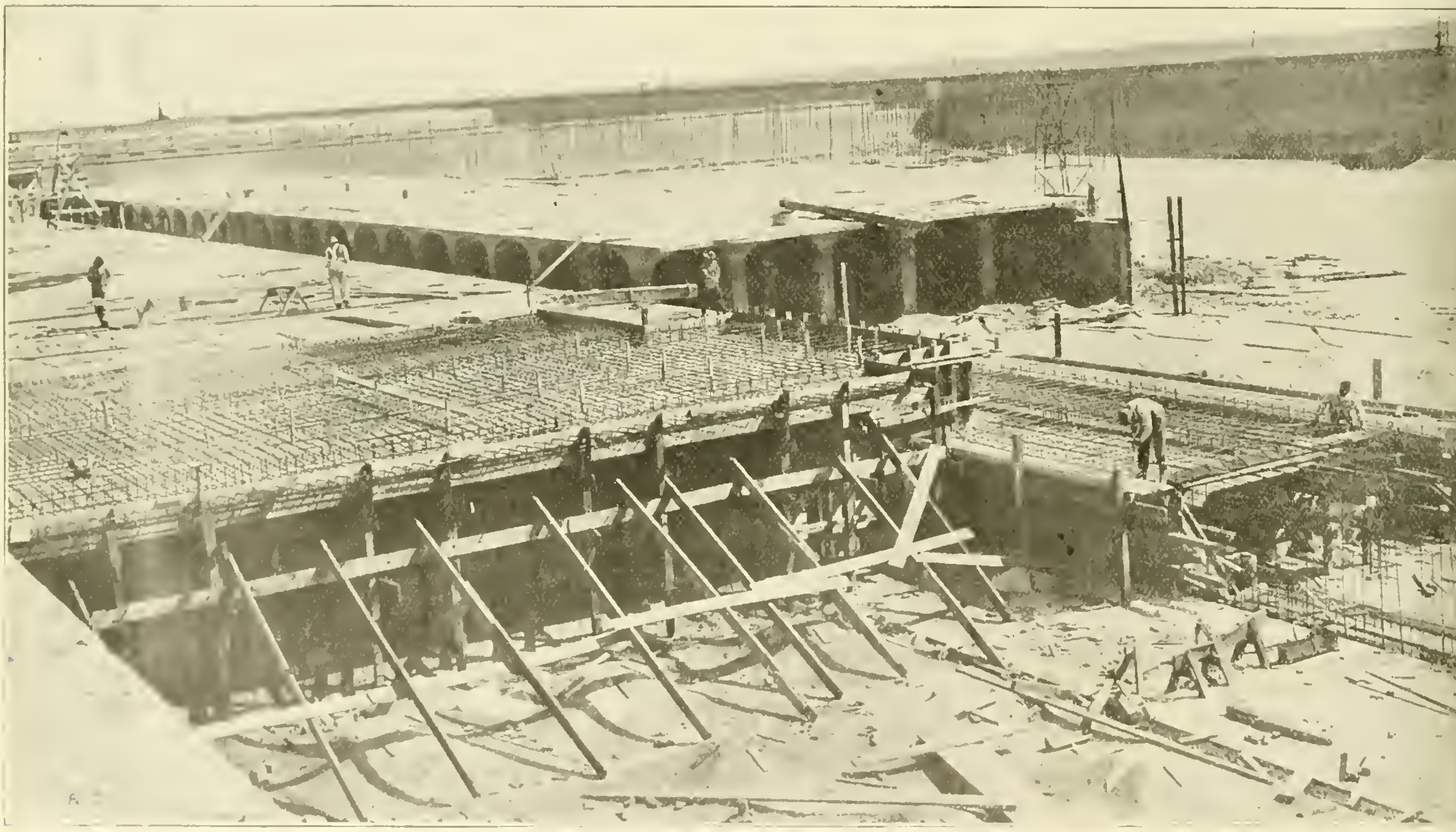


Fig. 2. Construction View of Substructure of New Mechanical Water Filtration Plant at St. Louis.

In 1903 the Water Commissioner, Mr. Ben C. Adkins, with the writer as his principal assistant, was appointed by Mayor Rolla Wells with the express understanding that before the opening of the World's Fair in May, 1904, some scheme must be designed and put in operation to clarify the water supply. It is unnecessary to enter upon any history or description of the experiments and method used during that year, since the fact has been well advertised and is generally known, especially among water works engineers, that the St. Louis water supply was clarified and purified without the use of filters and by a process of coagulation and sedimentation. Also it is equally as well known that the process has been in successful operation for about 10 years. About two years ago it became evident that the capacity of the purification plant in use was too small for the treatment of the quantity of water consumed by the city. The eight years of operation of the purification process had also forced all of us who had been connected with the installation and development of the process, to admit reluctantly that it was not as efficient as filtration; that there were difficulties of operation that had not been satisfactorily overcome; that there were objections that could not be altogether removed. Whether to increase the capacity of the system by perpetuating the method in use, or by utilizing the machinery and basins for the preliminary treatment of the water, and then passing it through rapid sand filters, became principally a question of economy in first cost and operation. Careful estimates on the cost of increasing the capacity to 150,000,000 gals. per day showed that to build additional basins, conduits, etc., to enlarge the present system would cost at least $250,000 more than to build a rapid sand filter plant to supplement the treatment in existing basins. It was also estimated that the cost of treatment and operation would be less with the filters. Considering also the unquestioned fact that the general quality of the water would be improved, and that a uniform clarity of water would be insured by filtration, there could be no hesitation in making a decision in favor of the construction of the rapid sand filters.

In January, 1912, the Water Commissioner was authorized to prepare plans and specifications for changes and additions to the existing works at the Chain of Rocks, including the filter plant, so as to bring the working capacity up to 150,000,000 gals. per day.

The general layout of the Chain of Rocks works, giving the location of the filter plant, head house, mixing chamber, conduits and connections, with reference to existing structures, is shown in Fig. 1.

The filters are located in the western portion of basin 7, occupying a space approximately 700 by 110 ft. The filters were located in the basins, rather than on ground outside for several reasons. For example, the space taken from the basin could be readily spared, since ample basin capacity for coagulation and clarification would remain, construction could proceed more rapidly since no excavation was necessary, better and more convenient arrangements for utilizing existing conduits and making new basin connections could be had, and on the whole the cost would be decreased by such a location.

DESIGN OF NEW WORKS.

The design provides for 40 filter units, each 50 by 28 ft. in plan, supported on concrete columns 20 ins. square, spaced 11 ft. centers north and south and 10 ft. 3 ins. east and west.

On the original basin floor inverted concrete arches were laid to distribute the foundation loads of the columns. The filter boxes rest on the reinforced concrete floor formed by leveling up the groined arches joining the tops of the columns. Each filter box is poured as a monolith, the concrete mixture being 1:2:4, with 25 lbs. of hydrated lime added for each barrel of cement, for the purpose of making the boxes water-tight. The concrete in the columns and arches is composed of a mixture of 1:2½:5. Screened and washed gravel has been used throughout for the coarse aggregate.

The strainer system consists of the channel type, in which the drains are formed by and between concrete ridges, and covered by perforated strainer plates anchored to the concrete by bronze bolts and bars. Along the center of each half of each filter unit is formed a collecting channel, 28 ins. wide by 5 ins. deep, into which all the small lateral drains 3 ins. wide by 5 ins. deep empty. Four 10-in. openings in the bottom of each collecting channel are connected by cast-iron pipe to the effluent conduit.

The water will be filtered through 30 ins. of sand and 12 ins. of gravel. The gravel will be placed on the filter bottom in three layers, the bottom layer of 5 ins. in depth composed of gravel, ranging in size from ¾ to 1½ ins. in diameter; the second layer 4 ins. in depth, size from ⅜ to ¾ in., and the top layer 3 ins. deep, size from 3/16 to ⅜ in. The sand will have an effective size from 0.40 to 0.50 mm., and a uniformity coefficient not greater than 1.65.

The gallery between the filters contains the effluent conduit 17 ft. wide and 4½ ft. high.

over which lies the 36-in. cast iron wash water pipe, and above that is the influent flume, 15 ft. wide and 10 ft. deep, into which the water is drawn at each end of the filter house from basins 7 and 8. Both the influent and effluent flumes are built of reinforced concrete. The cover of the influent flume will be the operating floor of the filter house. Each filter has a 24-in. connection to the wash-water pipe. All drainage and wash water from the filters is carried through 30-in. sewers to the main sewers and thence to the river.

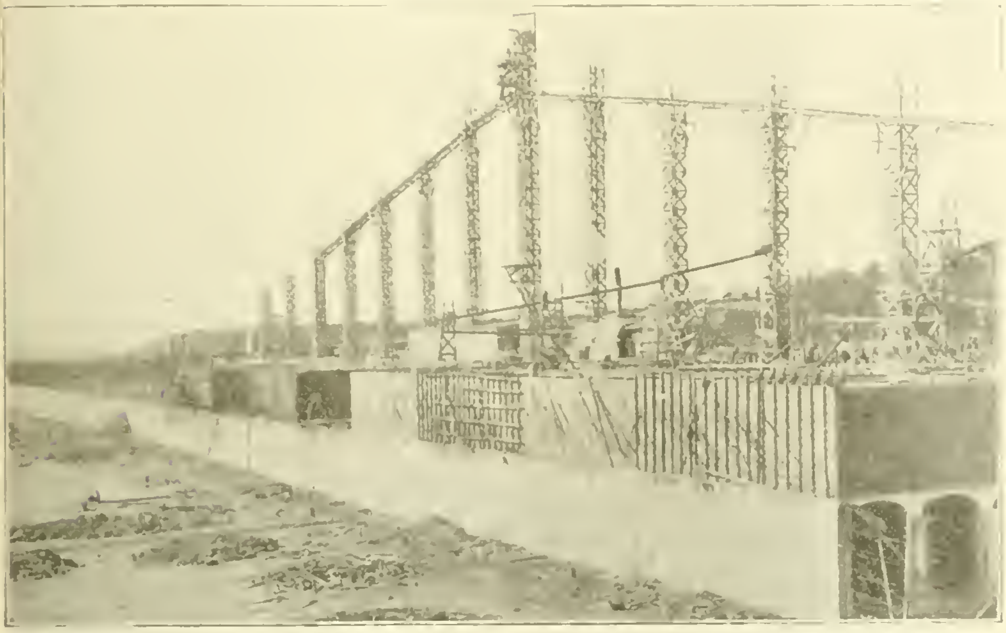


Fig. 3. Construction View of New Mechanical Filter Plant at St. Louis Showing Substructure, Filter Boxes and Concreting Tower System.

The rate of flow through each filter will be maintained by a rate controller placed on the 24-in. effluent pipe. Each controller can be set to maintain any desired rate of flow between 1,500,000 and 5,000,000 gals. per 24 hours, regardless of the difference in head on the two sides of the controller, which difference may amount to as much as 14 ft.

The registered flow must not vary more than 3 per cent from the actual flow below the 2,500,000 gals. daily rate, or more than 1½ per cent when the rate is above that amount. These 40 individual controllers will be connected to a master controller which when set at any desired rate of flow will automatically [illegible] each filter at the same rate.

The [illegible] of each filter will be [illegible] from an [illegible] in the central gallery. [illegible] valve levers, gauges and [illegible]. The 24-hour recorder [illegible] the loss of head [illegible] and the actual quantity of [illegible] through the controller [illegible] recording gauge [illegible] the [illegible] of the filter house.

Venturi meters [illegible] will be placed on the [illegible] and on the outlets of the [illegible]. These meters record [illegible] depend [illegible] water passing through the [illegible] may be [illegible] which are [illegible] meters.

The filter house [illegible] the filters, will be [illegible] window sash and frames [illegible] will be almost [illegible] at the side walls [illegible] the central gallery [illegible]. On the concrete roof [illegible] tar and gravel [illegible]. The building, which covers an [illegible] will be heated from [illegible] house. The current for lighting the filter and head houses and for operating motors will be furnished from a generator house, located at the south end of the pumping engine house. This generator house will be a brick building 58 ft. by 66 ft. conforming architecturally to the existing buildings at the pumping station. The machinery will consist principally of two 250-KW. steam-driven units.

The head house is to be a four-story concrete building located 34 ft. south of the filter house. On the first floor will be three 150-HP. tubular boilers, four alum solution tanks, two alum solution pumps, three 12-in. centrifugal pumps for filling the wash water tanks, two 6-in. pressure pumps for operating hydraulic valves, a four-ton refrigerating plant for manufacturing ice for the department, a store room for hypochlorite of lime and liquid chlorine, and a small machine shop. On the second floor will be located an office, a small laboratory, six alum mixing tanks, wash rooms, lockers and store room. Two wash water tanks, each 50 ft. in diameter and containing 175,000 gals. of water, will occupy the greater part of the third and fourth floors. The dome-shaped covers of these tanks will form a portion of the roof of the building. There will be [illegible] on the third floor [illegible] for hypochlorite of lime and [illegible] liquid chlorine [illegible]. On the fourth floor the space will be [illegible] hypochlorite mixing [illegible]. Alum [illegible] cars [illegible] will be [illegible] fourth floor.

[illegible]

tity of water being pumped, so as to maintain the velocity sufficiently high to prevent settlement in the channels.

The effect of the passage of the water through the mixing chamber is to insure the thorough mixing of the milk of lime with the water and also to allow a longer time for the softening action of the lime before the sulphate of iron is added. This will result in an estimated saving of probably 30 per cent in chemicals, because of the elimination of loss due to incomplete chemical reactions. The water on leaving the mixing chamber and entering the filling conduit, will receive the charge of sulphate of iron. The inlets from the filling conduit to each basin will be baffled to prevent direct flow across the basins. The water will be drawn from the primary basins into a concrete collecting conduit, located between the two sets of basins. From this conduit, it will be delivered and measured through two Venturi meters, with 56-in. throats, into a chamber 75 ft. wide, extending the full distance across the north end of basin 7. As the water enters this chamber, it will receive any secondary treatment that may be necessary before it is drawn into basins 7 and 8. The addition of hypochlorite of lime or liquid chlorine when necessary may be made just before the water goes to the filters or these chemicals may be applied to the filtered water as it leaves the filter house.

The rated capacity of each of the 40 filter units is 4,000,000 gals. per 24 hours, or 160,000,000 gals. per day for the plant. This output may be increased to 200,000,000 gals. per day, should it become necessary to supply this amount of water.

The contract for the concrete substructure, filter boxes, conduits and connections was awarded to the McCormack-Combs Construction Co. of St. Louis on April 18, 1913, for the sum of $225,023.10. Because of the location of the filters in basin 7, it was possible for the contractor to design a comparatively simple plant for mixing and placing concrete. A switch track was laid along the basin wall, so that material could be dumped from cars into bins above the mixer which

Fig. 4. View of Filter Boxes and Wash Water Line of New Filters at St. Louis, from Concreting Tower.

[illegible] surface of the [illegible] about [illegible] cars [illegible] shed will [illegible] into a skin [illegible]. From this [illegible] the entire length of the filters, a total of about 700 ft. Concrete can be taken from the main chutes at any one of the twelve towers and carried to any part of the work. The grade of the main chutes is 20 per cent. They are made of wood, lined with sheet iron, are 12 ins. in width and depth and span 45 ft. between towers. The moveable chutes are made of steel in one piece, and are 12 ins. wide by 10 ins. deep.

The velocity with which the concrete flows from the main tower to its destination is about 480 ft. per minute.

The mixing and placing of concrete was begun on July 17, 1913, and up to Feb. 1, 1914, a total of 165 working days, 14,428 cu. yds. of concrete have been poured. This includes the entire substructure, 38 filter boxes, the effluent conduit and the greater portion of the influent conduit. Twenty-four filter boxes were poured during 27 consecutive days in December and January, each box containing 107 cu. yds.

The cost of erecting form work for the walls, columns and arches was $1.65 per cubic yard of concrete, and for the effluent conduit $1.55 per cubic yard.

Following is the list of contracts let, and the estimated costs of the work necessary to complete the plant:

1. Filters, McCormack-Coombs Construction Co.	$ 225,023.00
2. Filter equipment, Pittsburg Filter Mfg. Co.	398,200.00
3. Filter house, McCormack-Coombs Const. Co.	89,730.00
4. Head house, estimated cost	185,000.00
5. Waterproofing Basins 7 and 8, estimated cost	50,000.00
6. Raising division walls, estimated cost	85,000.00
7. Generator house, estimated cost	50,000.00
8. Mixing chamber, estimated cost	160,000.00
Total	$1,243,953.00

Contracts for the last five items will be let within 60 days, and it is intended to have the plant ready for operation by Nov. 1. All contracts will require completion before that date under heavy penalties for overtime. This plant when completed will be the largest mechanical filter plant in the world.

Method and Cost of Repairing the Cedar River Wood Stave Pipe Line of the Seattle Water Works.

The following brief statement of the methods employed in repairing the Cedar River wooden pressure pipe line of the Seattle water works, and giving the corresponding actual field costs has been compiled from information furnished by Mr. L. B. Youngs, superintendent of the Seattle water department.

Cedar River Water Supply Pipe Line No. 1 was put in commission in January, 1901. The pipe is mainly 42 ins. inside diameter, though some parts are 44 ins. It is built of 6-in. staves made from the native Douglas fir. These staves are cut from 2-in. x 6-in. scantling dressed outside and inside to true circumferential, and on the edges to radial lines.

After 13 years the steel bands do not seem to be seriously corroded. The staves, however, or more accurately speaking, individual staves here and there, began to show evidence of serious decay as early as seven years after installation. Other staves right alongside of them have remained practically sound. It has been necessary, therefore, to renew certain staves, rather than to renew the pipe as a whole. At some places where the pressure is light and where the covering is of loose gravel which readily admits the air and changes of temperature sections from a few hundred to a few thousand feet have been fully replaced, after a use of 12 years, with new pipe, using, of course, the old steel bands.

The method of repair is simple. It consists of uncovering the pipe, loosening the bands, taking out the decayed stave, inserting the new stave in its place, and cinching the pipe up again.

After the staves have been planed to form in the mill they are 1⅝ ins. thick. These staves will hold water very often until in only a shell ¼ in. thick. Of course this is where the pressure is comparatively light, and the backfilling compact. Sapwood naturally decays rapidly and the department specifies that not more than one-fourth of the thickness of the stave, and then only on the inside, shall be sap. The accompanying view shows the state of two of these staves that have been replaced.

Following is a detailed statement of the actual cost of replacing 1,600 lin. ft. of 44-in. pipe in 1913. Common labor was $2.75 per day.

The work was done by day labor under departmental direction.

COST OF REBUILDING 1,600 FT. OF 44-IN. WOOD STAVE PIPE AT SEATTLE, WASH., IN 1913.

Cost of Labor:

Item:	Amt.	Cost.	Cost per lin. ft.
Excavation—			
130 days labor	$ 357.50		
28¼ days team	141.25	$498.75	$0.31
Wrecking pipe—			
30 days labor	82.50	82.50	0.05
Painting staves—			
17 days labor	46.75	46.75	0.03
Erecting pipe—			
72 days labor	198.00	198.00	0.12
Cinching pipe—			
58 days labor	159.50	159.50	0.10
Tamping pipe—			
78 days labor	214.50		
26 days team	130.00	344.50	0.22
Total cost of labor, 1,600 ft. pipe	$1,330.00		

Total cost of labor, 1 ft. pipe, $0.83.

View of Wood Staves Removed from Cedar River Pipe Line, Seattle, Wash.

Cost of Material:

Item:	Amt.	Cost.	Cost per lin. ft.
42,551 ft. stave lumber at $31.25 per M	$1,329.72	$1,329.72	$0.83
Hauling staves—			
6 days labor	16.50		
13¾ days team	68.75	85.25	0.05
155 gals. C. A. wood preserver at 65 cts.	100.75	100.75	0.06
Total cost of material, 1,600 ft. pipe (exclusive of iron bands and shoes)	$1,515.72		

Total cost of material per lin. ft. of pipe	$0.95
Total cost of labor per lin. ft. of pipe	.83
Total cost of pipe rebuilding per lin. ft.	$1.78

The painting mentioned under labor is an experiment. The department officials cannot say just what its effect will be.

The Relation of "Out-of-Pocket Cost" to Water Rate Making, With an Illustrative Example.

Numerous discussions by experts of such topics as valuation, depreciation, rate of returns, etc., extending over a period of several years, promise a substantial agreement on some of the perplexing problems affecting the determination of a fair *gross* return for a public utility corporation. It should not be forgotten, however, that the determination of such a *gross* return is only a preliminary means of just and equitable rates. While a more perfect agreement is being reached by experts as to some of the subjects above named, it is well to take up simultaneously some of the unsettled points in the construction of rate schedules. Attention was called to this fact by Messrs. Morris Knowles and Maurice R. Scharff, consulting engineers of Pittsburgh, Pa., in their paper before the recent annual meeting of the Illinois Water Supply Association. In their paper they discussed briefly the relation to rate making of one element which they call "out-of-pocket cost." The treatment of the subject refers to water rates although a similar line of reasoning is equally applicable to some other public utilities, and to some measure applicable to all. The major portion of the paper follows:

Owing to the difficulty of recognizing inequalities in rate schedules and to the relatively satisfactory results attained by the customary unscientific methods of framing them, there has not been so much study given to this portion of the general problem of valuation and rate making as to some others. And yet, all who give the matter thought, must recognize it as one of the most important and complex portions of that problem. Indeed, so complex is it, involving as it does many yet unanswered questions of accounting and of commercial and economic policy, and touching at many points the human element, which cannot be mathematically estimated, that it may well be said to be incapable of exact solution.

The perfect system of rates cannot be determined *a priori*; it must come, rather as the result of evolution, of gradual improvement as the outcome of human experience. It is, however, possible, with an understanding of such principles as have already been worked out, and using the best judgment we can bring to bear to attain results far preferable to those based on the belief that it is useless to compute rates at all. For the former method is bound to reach by experience a closer and closer approximation to the truth; while the latter can never advance a single step.

Now in spite of the complexities and uncertainties in the computation of reasonable rates, their definition is fairly well agreed upon. Reasonable rates have been defined as

> Such as will, in any efficiently operated utility, yield sufficient revenue to permit satisfactory service, and to pay in addition to operating and maintenance expenses and depreciation a fair return upon the fair value of the property used and useful for the public service—all without discrimination and with due regard for the public interest.

Fortunately, we are not concerned at this time with the meaning of satisfactory service, reasonable expenses, depreciation, fair return, or fair value. We may, therefore, assume them all satisfactorily settled, and the proper tion to lower terms in somewhat the following form:

> Reasonable rates are such as will assess the proper gross income upon consumers without discrimination, and with due regard for the public interest.

With these preliminary remarks, let us consider the meaning of the terms "without discrimination" and "with due regard for the public interest."

TABLE I. ALLOCATION OF ESTIMATED TOTAL REVENUE.

Item.	Total.	Pct. division.		Division		Pct. division readiness to serve.		Division readiness to serve.		Pct. division output.		Division output		Apportionment			
		Readiness to serve.	Output	Readiness to serve.	Output	Capacity.	Services.	Capacity	Services	Out of pocket	Remaining output.	Out of pocket.	Remaining output	Capacity	Services.	Out of Pocket.	Remaining Output.
Advertising	$ 200	100		$ 200.00		100		$ 200.00						$ 200.00			
Insurance	150	100		150.00		100		150.00						150.00			
Interest	29,190	100		29,190.00		100		29,190.00						29,190.00			
Supplies	850	28	72	240.00	$ 610.00	46	54	110.00	$ 130.00		100		$ 610.00	110.00	$ 130.00		$ 610.00
Legal	450	100		450.00		100		450.00						450.00			
Light	60	100		60.00		100		60.00						60.00			
Meetings	225	100		225.00		100		225.00						225.00			
Patroll'g res'v's	600	100		600.00		100		600.00						600.00			
Pumping:																	
Engineer	800		100		800.00						100		800.00				800.00
Coal	2,400		100		2,400.00					100		$2,400.00				2,400.00	
Incidentals	300		100		300.00					10	90	270.00	30.00			270.00	30.00
Power	1,000		100							100		1,000.00				1,000.00	
Repairs:																	
General	200	100		200.00		20	80	40.00	160.00					40.00	160.00		
Reservoirs	200	100		200.00		100		200.00						200.00			
Pumps	3,000		100		3,000.00					20	80	600.00	2,400.00			600.00	2,400.00
Filters	700		100		700.00					20	80	140.00	560.00			140.00	560.00
Distribution	3,000	50	50	1,500.00	1,500.00	100		1,500.00		10	90	150.00	1,350.00	1,500.00		150.00	1,350.00
Rent	300	100		300.00		50	50	150.00	150.00					150.00	150.00		
Salaries:																	
Gen. manag'm't	2,500	100		2,500.00		10	90	250.00	2,250.00					250.00	2,250.00		
Opr. manag'm't	3,500	50	50	1,750.00	1,750.00	80	20	1,400.00	350.00		100		1,750.00	1,400.00	350.00		1,750.00
Telep. & telegr.	200	100		200.00		50	50	100.00	100.00					100.00	100.00		
Traveling	300	100		300.00		100		300.00						300.00			
Teams	400	50	50	200.00	200.00	100		200.00		10	90	20.00	180.00	200.00		20.00	180.00
Taxes	2,100	100		2,100.00		100		2,100.00						2,100.00			
Analyses	150	50	50	75.00	75.00	100		75.00			100		75.00	75.00			75.00
Bleach	35		100		35.00					100		35.00				35.00	
Filter oper't'n:																	
Labor	500		100		500.00						100		500.00				500.00
Alum	500		100		500.00					100		500.00				500.00	
Incidentals	200		100		200.00					50	50	100.00	100.00			100.00	100.00
Depreciation:																	
Reservoirs	3,965	100		3,965.00		100		3,965.00						3,965.00			
Pumping stat'n	1,304	30	70	391.20	912.80	100		391.20		14	86	130.40	782.40	391.20		130.40	782.40
Filter	633	50	50	316.50	316.50	100		316.50		20	80	63.30	253.20	316.50		63.30	253.20
Distribution	2,228	90	10	2,004.80	222.80	100		2,004.80			100		222.80	2,004.80			222.80
General	325	100		325.00		50	50	162.50	162.50					162.50	162.50		
Reading meters	500	100					100		500.00						500.00		
														$44,140.00	$3,802.50	$5,408.70	$9,613.40

"Discrimination" is defined in the Standard Dictionary as "differential treatment," and the literal application of this definition would seem to require that rates "without discrimination" should be based strictly on cost of service, without differential treatment according to amount of use, hours of use, kind of use or any other distinction. It has been repeatedly decided, however, that this term does not preclude the classification of consumers and differential treatment of the classes, on any reasonable basis. Some extension must therefore be given to the literal meaning of the term, in order to make it workable, and while it has not been clearly formulated by the authorities, the "rule of reason" would appear to require that discrimination be defined as "differential treatment, except in so far as public interest requires."

Pursuing this line of thought a step further we have come to the conclusion that there are but two grounds on which "public interest" may require differential treatment of consumers, namely, first, in order to secure some contribution to the public welfare (a method of indirect taxation of those against whom the differential treatment militates) and second in order to lower the cost to other consumers by retaining or securing custom which would otherwise be lost.

Some will no doubt contend that "with due regard for the public interest" has reference to the general welfare, and that it may justify the lowering of rates for certain necessities to certain consumers even to below cost, for the sake of extending the benefit connected with their use, or in order to encourage business, or for some similar reason. The argument appears to be that, while such steps have the effect of throwing a disproportionate share of the expense upon other services, or upon other consumers, it is nevertheless justifiable as a kind of indirect tax, and as in the public interest. Thus it may be held that freight rates on coal should be low so as to permit the cheap movement of fuel, and to prevent the localization of industry near the coal fields, with resultant evil social effects. Again, it is argued that water should be supplied free to charitable institutions and to new factories or industries, on account of the public interest therein; and that water should be supplied to small consumers at excessively low rates, so as to favor liberal use and promote public health. But we see no force in these arguments as applied to rate making. Indirect taxation is a useful part of our system of raising govermental revenue, and on account of the relative ease of administration and collection, may well be retained until equitable methods of direct taxation, such as income taxes, are more thoroughly worked out. But we are inclined to believe that its use should be confined to raising public revenue and that it should never take the place of actual governmental appropriation as a method of subsidizing any industry or service, no matter how desirable the encouragement of the latter is from the point of view of public welfare. We believe that President Wilson is deserving of the highest commendation for refusing to compromise with a principle in order to carry out an ill considered plank of the Baltimore platform, and for demanding of Congress that if coastwise shipping is to be subsidized, it be done in the open and above board by congressional appropriation, and not by the underhanded method of remitting tolls for passage through the Panama Canal.

DEFINITION OF "OUT-OF-POCKET" COST.

There remains, then, only the second basis which seems to us proper as a reason for differential treatment of consumers—in order to lower the cost to other consumers by retaining the custom which would otherwise be lost. How this works out may be very readily explained. Assume a "model" water works with a gross income, which must be collected when it is supplying 1,000,000 gals. per day, of $35,000. Now assume one large consumer using 100,000 gals. per day and paying $3,500 per year should find that he could secure water elsewhere at a cost of $2,500 per year, and should disconnect from the mains. The amount supplied would drop to 900,000 gals. per day, but interest, depreciation and maintenance would remain practically the same. It would still be necessary to have the same number of engineers and firemen, and the only saving would be a small amount on fuel, alum, hypochlorite, oil and waste, etc., which would certainly not be in excess of $1,000 per year, and would probably be a great deal less. So the gross income to be raised might be reduced to say $34,000, all of which would have to be raised from other consumers. Now if all other conditions remained the same, and the rate for the large consumer were reduced so as to just hold his custom, when paying $2,500 per year, the total gross income would still be $35,000, and the portion to be raised from consumers other than the large one would be $32,500 or $1,500 less than in the other case. Their rates could, of course, be correspondingly lower than would be necessary if the large consumer were lost. The advantage of such an arrangement to all of the consumers is obvious, and such differential treatment if necessary to secure this advantage, would not only not constitute discrimination as we have defined it, but would appear to be a plain duty of the water works as a public utility.

In brief, it is to the advantage of all consumers that large, long hour, high load factor consumers be retained. Differential treatment of them does not constitute discrimination, so long as it will result in advantage and saving to other consumers. Such concessions in rates, however, must not be greater than actually necessary to meet the competition of other sources of supply. And in any case they must not be so great as to increase instead of decreasing the income to be collected from other consumers. Thus in the case cited above, if it were possible to get the business at $2,500 any rate less than that sum would constitute discrimination. While if it were really necessary in order to meet competition any rate greater than $1,000 per year would effect some saving for other consumers and would be reasonable. A rate less than $1,000, however, would make the amount to be collected from other consumers greater than if the large consumer ceased entirely to take service. This would certainly be discrimination under our definition.

The lower limit ($1,000 in the above case) below which rate concessions cannot be made, even if necessary to secure business without constituting discrimination, and which represents the actual expense which would be saved if the consumer in question were disconnected, we have called the "out-of-pocket cost." We have found its estimation of value, not only for determining the lower limit of the proper total annual charge, but also practically, in framing rate schedules.

The cost of supplying water may be said to be made up in the first analysis, of the cost of readiness to serve, or costs which result from providing the plant and standing ready to serve and which are wholly independent of the amount of water supplied; and output costs, which result from operating the plant, and which vary more or less closely with the amount of water supplied.

The cost of readiness to serve may be further sub-divided into capicity cost, of which interest is the best example, which varies directly as the capacity of the plant, and may equitably be distributed to consumers in proportion to their demands, which together require this capacity; and the service cost, which varies with the number of services, and may equitably be assessed equally upon all services. Meter reading, billing and collecting are typical of this.

Similarly, the output cost may be divided into two parts, the out of pocket cost, defined as above, and including all expenses for materials and labor used in small units and varying closely with the consumption such as coal and alum; and the remaining output cost, which includes expenses occurring in large units and varying somewhat with, but not directly as the consumption. Typical of these are the wages of pumping engineers and firemen.

In computing a rate schedule, it seems reasonable to divide the gross income into these four parts, and to combine the demand charge, equal to the demand of the consumer (as measured by the size of meter) over the total demand, times the total demand cost, and the consumer charge, equal to the consumer cost, divided by the number of consumers, in a minimum charge. If no differential treatment were necessary, all the remaining costs would then be assessed by a uniform meter rate, obtained by dividing the total output cost by the total annual metered sale of water. Differential treatment, in order to secure the business of favorable consumers who would not take water under these circumstances, might then be effected by relieving such consumers either of a portion or all of the minimum charge, or of a portion or all of the remaining output cost, or of a portion or all of both. But it happens that the minimum charge so calculated is ordinarily a small part of the cost of water to the large, long hour, highload factor consumer to whom concessions must be made. Moreover, it would unduly complicate rate schedule computations to attempt to divide the necessary concession between the minimum charge and the meter charge. And we have not, in our experience, met with any such case in which the possible concession in the meter rate was not sufficient or more than sufficient to get the business.

Conceivably it might be possible to classify consumers into groups by quantity used, location, or whatever reasonable basis made differential treatment necessary, determining for each group the cost of an alternative supply to which they would turn if concession were not made, and fixing in this way the extent to which it would be necessary and proper to relieve each group of a portion of the charge. But ordinarily this would be difficult or impossible, and it will usually be preferable to group all such large consumers, whose loss is threatened, into one group, determining the extent to which it will be necessary to relieve the consumer most favorably situated with respect to an alternative supply and either apply his rate to all large consumers, or if he is the largest (as is likely) taper the rate to this in uniform steps from the smaller consumer's rate.

These computations cannot, of course, be established as theoretically precise, but improved accounting and good judgment will lead to results in which confidence may be had as closely approximating the ideal. As an example of the method, we present below in skeleton the computation involved in its application to a rate schedule prepared for a small water company in the East.

TABLE II.—COMPUTATION OF DEMAND CHARGES.

Size of meter. ins.	Nos. of meters.	Demand per meter. c. f. p. m.	Total demand.	Demand charge per meter.
5/8 1 spigot	500	1	500	$ 4.00
5/8	1,909	2	3,818	8.00
3/4	220	4	880	15.90
1	109	8	872	31.80
1½	1	12	12	47.80
2	1	20	20	80.00
3	8	40	320	160.00
4	4	80	320	320.00
6	3	120	360	480.00
8	2	160	320	640.00
Total	2,757		7,422	

The fair normal gross income for the period to which the rates were to apply was first determined as $62,965. This was then distributed in the manner shown in Table I.

The fair fire protection payment to be made by the city was then computed as $14,566 (the method need not concern us at this time). Deducting this from the total capacity cost, we have $29,574 left to assess on consumers as demand charges.

The estimated number of meters and the assumed demand of each having been agreed upon, the demand charge for each size of meter was then computed as shown in Table II.

The service charge was then computed by dividing $3,802, the total service cost, by the total assumed number of meters, and was equal to $1.40.

The total annual metered consumption was then estimated at 452,500,000 gals., of which 270,000,000 gals. would be taken by a single large industry—the one consumer, if any, to whom concession would have to be made. Dividing the total output cost by the total consumption, we arrive at $0.033 per 1,000 gals. as the average meter rate at which all water would be sold if no differential treatment were necessary. Dividing the out of pocket cost by the total consumption, we have $0.012 per 1,000 gals., as the out of pocket meter charge, or the lower extreme to which the meter rate of the large consumer could properly be reduced, if the maximum concession had to be made. And dividing the remaining output cost by the consumption of all except the one large consumer, and adding the quotient, $0.053, to the out of pocket meter charge, $0.012, we get $0.065 per 1,000 gals. as the meter rate which small consumers would have to pay if the maximum concession were made to the largest consumer.

In any particular case of this kind, the next step would be to determine to what extent concession would be necessary in order to retain the custom of the large consumer, and so to fix between these limits the rates for the several classes of service. It happened in this case that it was determined that there would be no danger of losing the large consumer even at the average meter rate, so that the latter was recommended for adoption. It was also considered advisable in this case to include in the minimum charge, for ⅝-in. meters, only a charge, at the average meter rate, for 30,000 gals. per year, in order to forestall the criticism that metering would curtail the use of water by the poor to a point below the minimum requirements of hygiene. The resulting schedule of net charges, together with the gross rates made necessary in order to apply a 10 per cent discount for prompt payment, and the rounded figures recommended for actual use, are shown in Table III.

TABLE III.—NET, GROSS, AND ROUNDED RATES COMPUTED AS ABOVE.

Item.	Size of meter. ins.	Net charge.	Gross charge.	Rounded rate.
Minimum charge*.	5/8†	$ 6.40	$ 7.10	$ 7.00
Minimum charge*.	5/8	10.40	11.60	12.00
Minimum charge..	3/4	17.30	19.20	20.00
Minimum charge..	1	33.20	36.90	40.00
Minimum charge..	1½	49.20	54.70	55.00
Minimum charge..	2	81.40	90.40	90.00
Minimum charge..	3	161.40	179.30	180.00
Minimum charge..	4	321.40	357.10	360.00
Minimum charge..	6	481.40	534.90	540.00
Minimum charge..	8	641.40	712.70	720.00
Meter	—	.033	.037	.04

*Minimum charge for 5/8-in meter only includes charge for 30,000 gals. of water per year.
†1 spigot.

Rivers and Harbors

Flood Prevention and Protection From the Point of View of the Federal Government.

During the period of great floods last year, the Secretary of War directed the Chief of Engineers, U. S. Army, to appoint a board of officers to make a thorough inquiry and examination into existing conditions and "report upon the most practicable and effective measures for prevention of damage by floods to works constructed for the improvement of navigation, of interference with interstate commerce and of other disastrous results thereof." As a member of the board created in accordance with that order, Major J. C. Oakes, Corps of Engineers, U. S. A., presented a paper before the Indiana Sanitary and Water Supply Association which we reprint slightly condensed as follows:

I wish to take this opportunity to affirm that I am not advocating one method of protection to the exclusion of any other, but wish to show that all methods should be used as circumstances dictate; and above all, I wish to emphasize the immensity of the problem of flood protection for a large river system like the Mississippi, or even one of its main tributaries, such as the Ohio. I also desire to acknowledge my indebtedness for information to an unpublished report of the Board on River Floods.

FLOODS AND FLOOD PREVENTION MEASURES

Before discussing the government's attitude toward this question, I desire to discuss the subject of floods and the methods of prevention now commonly under discussion.

Floods are caused by excessive rainfall, quick melting of a thick blanket of snow or a combination of the two, quick runoff, and channels inadequate to take care of the resulting flow of water. The runoff may be hastened by frozen ground, or a ground saturated by previous rainfall so that all of the precipitation escapes rapidly. The natural channel capacities may be and usually are reduced by artificial obstructions which prevent free and easy runoff.

Flood prevention measures must reduce precipitation, retard runoff, or increase the capacity of the channels. I think that it is generally agreed that it is beyond the possibility of human ingenuity to control precipitation. The only method that has been seriously advanced is by forestation, and even for that method I am not aware that its advocates claim that forests decrease precipitation. It is certain that for all practical purposes the control of precipitation by forests or other means is at present impossible. Flood prevention must deal, therefore, with the other causes of floods.

RETENTION OF PRECIPITATION IN GROUND.

Runoff may be retarded by the retention of precipitation on or in the ground, or by the use of reservoirs. Several means of holding the precipitation where it falls have been proposed, such as deep plowing, contour plowing and forestation. The term deep plowing is self-descriptive and the method is advocated not only for the purpose of retarding runoff, but also as a means of improving the yield of agricultural land. If all cultivated land could be plowed deep and the material well broken up, a considerable amount of precipitation could be retained by the soil. However, as can be readily seen, as a means of flood prevention, it is hardly within the control of either the federal government or the states. Furthermore, with a storm like that of last March and April, where in certain localities in Ohio over 11 ins. of precipitation occurred, it is very doubtful if more than a very small portion could be held in the soil, even if plowed deep. Again, a large part of the country is uncultivated and covered by forests, hillsides and towns. This method, therefore, is very limited in its application and apparently impossible of control.

Contour plowing is a term applied to the method of plowing which creates ridges following the contours of the land and forms, as it were, terraces in such a manner that water will be held where it falls. This method, or perhaps a modification of it by the use of small dikes, is used extensively in foreign countries, such as Japan and the Philippines, where there is an excess of rain in one season to be followed later by a season of drouth. Under such climatic conditions, holding the water on the lands becomes necessary, else no crop can be raised.

In the eastern and middle western states these measures are not necessary for agricultural purposes, and while, if used extensively, they would retard runoff to some extent, they are impossible of control.

Both of these methods, therefore, seem to have advantages within certain limits, and would seem desirable, but they are not feasible means.

FORESTATION.

This brings us to forestation. This subject was treated at some length in the report of the National Waterways Commission, and the following is quoted from their report:

> It is generally admitted that forests exercise such reservoir characteristics and under favorable conditions to a sufficient extent to improve the regularity of stream flow. There is, however, a decided limit to the quantity of water which a forest cover can absorb. The capacity for absorption varies greatly under different conditions, depending upon the depth and character of the forest litter, as well as of the soil underneath, whether pervious or impervious, also upon the condition of the ground, whether frozen or not, upon the steepness of the slope, and numerous other factors. Where the forest litter is destroyed by forest fires, or is removed to prevent them, the absorptive capacity is thereby reduced.
>
> Various experiments have been made to ascertain the amount of water which different kinds of forest litter could absorb and hold. The results show that in general an amount equal to a precipitation of 0.16 in. can ordinarily be retained, while under favorable conditions the absorption of an amount equal to 0.24 in. or even more is possible. The soil beneath the humus may also be capable of some absorption. As soon as the saturation point is reached additional rainfall must necessarily run off at the surface just as if the ground were deforested. This explains why forests are powerless to prevent floods, although to the extent that they absorb the precipitation [illegible] diminish them.
>
> Effects of Forests Upon [illegible] [illegible] caused primarily by a heavy [illegible] precipitation, amounting often [illegible] to several inches within 24 hours. During the [illegible] which caused the disastrous [illegible] in the Passaic valley in October, 1903, [illegible] according to records taken at New York [illegible]. The worst floods [illegible] when these heavy rains fall upon a considerable accumulation of snow, [illegible] augments the amount of [illegible] precipitated. At this time the ground [illegible] be frozen or saturated and [illegible] absorption to that extent [illegible].
>
> Forests retard the [illegible] in the spring, and by allowing [illegible] from this source to be absorbed [illegible] influence upon stream [illegible] spring rains fall upon [illegible] and cause it to melt [illegible] the effect of the forest is in such a case to aggravate rather than ameliorate flood conditions. It thus appears that under one set of conditions forests may exercise a beneficial influence upon stream flow and floods, while under another their influence would be harmful.

Assuming that the above is a fair statement of the effects of forests on stream flow, it will be noted that an amount of precipitation equal to 0.16 in. can ordinarily be retained, while under favorable conditions the absorption of an amount equal to 0.24 in. is possible. Applying the factor of 0.16 in. to the state of Ohio, and to the storms which created the flood of March and April, 1913, if the whole state of Ohio could be reforested and could retain 0.16 in. of rainfall, only 2⅔ per cent of the total rainfall would have been retained by the storage effect of this forest. The present forest area of Ohio is about 9,000 square miles, or a little less than one-quarter of the state of Ohio. If we could change an additional 10,000 square miles of present cultivated land back into forest, this additional forest under the most favorable hypothesis would retain only ⅔ of 1 per cent of the rainfall of such a storm as that of March 23-27, 1913.

This subject was very thoroughly discussed in the Transactions of the American Society of Civil Engineers in connection with an article by Brigadier-General H. M. Chittenden, Corps of Engineers, U. S. Army, on the subject of "Forests and Reservoirs in Their Relation to Streamflow, with Particular Reference to Navigable Rivers." As illustrating the bad influence that forests may have upon runoff, he states:

> In the first place, forests break the wind, prevent the formation of drifts and distribute the snow in an even blanket over the ground. * * *
>
> The water from the first melting of the snow blanket does not sink into the ground, but into itself. The forest shade thus holds the snow, which gradually becomes saturated from its own melting until the heat and warm rains of late spring and early summer arrive. * * *
>
> The result is that when the final melting begins the whole body of snow disappears very rapidly, rushing from every direction into the streams, swelling them to their limit and often causing disastrous freshets. * * *
>
> The delay in melting caused by the forest shade has simply operated to concentrate it into a shorter period and increase the intensity of the resulting freshet.

I believe that all of us will agree that the above statements are correct under certain conditions, and that by retarding the melting of the snow, forests have, on occasions, contributed to disastrous floods. Furthermore, even if it could be proven that forests exert on the whole a very beneficial effect, it is impracticable to convert the agricultural land into forests. This country is certain to be more thickly settled than at present, greater crops will be needed for the support of the people and all tillable land will be ultimately used for the cultivation of crops until timber becomes so scarce that its value will pay the farmer to set aside a portion of his farm for its cultivation. Another objection to this method is that the formation of a humus sufficient to have its maximum effect in absorbing rainfall requires a long period, probably at least a century.

For these reasons it seems to me inexpedient to advocate reforestation as a means of prevention of floods.

RESERVOIRS.

The last mentioned and most feasible method of retarding runoff is by the use of reservoirs. We have heard much on this subject within the last few years, and it has become a popular remedy for the prevention of floods.

If a certain amount of water [illegible] stream creates damage during its flow, it is apparent to anyone that if that water can be held in a reservoir and allowed to [illegible] no damage would occur. Therefore, without understanding the immensity of the problem, the ordinary man thinks he [illegible] remedy for floods and proceeds to become an advocate of what has come to be known as the reservoir method of flood prevention.

I desire to dwell on this subject at some length, for the purpose of showing that the problem is not as simple as stated above. There are without doubt many localities that may be protected by a reservoir or reservoirs. For instance, in the study of flood protection for the city of Columbus, Ohio, the engineers were able to show that there were available sites immediately upstream from that city, which could be converted into holding reservoirs sufficient to take care of a flood similar to that of March and April, 1913. If such reservoirs had been in operation and had been controlled properly, releasing only sufficient water to fill the channel and not overflow the banks, no serious damage would have occurred in Columbus. It is also reported that the consulting engineers for the city of Dayton have shown that not only that town but several others in the immediate vicinity may be protected in like manner. The Pittsburgh Flood Commission has studied the subject in connection with flood protection for their city, and they have evolved a plan which appears to be ample to provide against future disastrous floods at Pittsburg, if, however, the control of the reservoirs is possible to obtain results in accordance with the plans.

When one comes, however, to consider the prevention of floods in a river system like that of the Mississippi, the problem becomes exceedingly difficult. In the first place, reservoir sites are not generally found close to the points where the damage will occur, and this is particularly true of the Mississippi.

Colonel C. McD. Townsend, Corps of Engineers, U. S. Army, president of the Mississippi River Commission, in an address before the Drainage Congress in the spring of 1913, stated:

> To have retained the Mississippi flood of 1912 within its banks would have required a reservoir in the vicinity of Cairo, Ill., having an area of 7,000 square miles, slightly less than that of the state of New Jersey, and a depth of about 15 ft., assuming that the reservoir was empty when the river attained a bankful stage. Cairo is the logical location for a reservoir to regulate the discharge of the lower Mississippi. It will not only control the floods from the Ohio, but also the discharge from the Missouri and upper Mississippi. But if the reservoirs be transferred from the mouths of the tributaries to the headwaters, their capacities must be largely increased.

It is not to be supposed that the people of Illinois and Missouri would be willing to have a portion of their territory as large as the state of New Jersey turned into a reservoir.

In this connection it should also be remembered that at the headwaters of the Mississippi there is the largest artificial system of reservoirs in the world, with a capacity of 93,000,000,000 cu. ft. These reservoirs have been successful in slightly increasing the low water discharge of the Mississippi River above St. Paul and also in reducing floods in that portion of the river, but a hundred miles farther downstream it is impossible to detect their effect during either high or low water.

The Ohio River, in which we are more particularly interested, has a very large flood discharge. At Louisville, for instance, the maximum discharge is 790,000 cu. ft. secs. At the mouth of the Ohio the estimated maximum discharge amounts to 1,500,000 cu. ft. secs. Let us see what effect the system of reservoirs proposed for the protection of Pittsburgh would have had at Louisville during the flood of March-April, 1913. The Pittsburg Flood Commission proposed the construction and operation of seventeen reservoirs whose total capacity is approximately [illegible],500,000,000 cu. ft. The amount of water flowing past Louisville during the day of maximum height was approximately 67,992,000,000 cu. ft. The proposed reservoir system, therefore, would have been more than filled by one day's flow at Louisville. If we assume that the dangerous flood height at Louisville is 51 ft., then, to have kept the river below that height during this flood a storage capacity 7,300 square mile feet, equal to 200,000,000,000 cu. ft. would have been required, or over three times the capacity of the proposed Pittsburg system.

If there had been a reservoir just above Louisville, with a capacity of 7,300 square mile feet, and if such reservoir had been empty when the flood stage of the Ohio River occurred, the Ohio River at Louisville could have been kept below the flood stage. But anyone can see that it would not be feasible to construct a reservoir of such capacity between Pittsburg and Louisville because of the great value of the land, towns, improvements, etc., that would be submerged.

Another illustration may be taken from the Wabash River. The office of which I have charge had just completed the field work of a survey of this stream between Terre Haute and the mouth at the time of the 1913 flood. At Mt. Carmel, 95.7 miles from the mouth, discharge records had been kept and discharge curves had been constructed, and while no actual measurements were taken at the crest of this flood, assuming that the curves prolonged are correct above actual measurement points, we are able to arrive at an approximate value for the discharge of that stream at Mt. Carmel during the period of flood. To have reduced the flood height 1 ft. during 10 days would have required a reservoir capacity of 26,000,000,000 cu. ft., or a reservoir approximately 100 square miles in area, with an average depth of 10 ft. To have kept the Wabash River within its banks at Mt. Carmel between the date of March 25 and April 21 would have required a storage capacity of 260,000,000,000 cu. ft., amounting to 9,300 square mile feet. This would have required a reservoir or reservoirs having an area of 930 square miles with water at average depth of 10 ft. It is apparent to anyone who knows the territory along the Wabash River that it would be impossible to find any such reservoir site in the vicinity of Mt Carmel, and even if found, the cost of such site would be prohibitive.

It is evident, therefore, that on the large rivers, like the Mississippi and the Ohio, there will be few reservoir sites close to the main streams that can be used to control floods and that sites must be found on the tributaries and generally at their headwaters. This complicates the problem and makes necessary large numbers of reservoirs of immense capacity. Speaking of this point with reference to the Mississippi River, Colonel Townsend stated:

> When, on April 2, the gauge at Cairo attained a height of 54 ft., there was flowing down the Mississippi River at least 2,000,000 cu. ft. of water per second. It requires about 11 days for a flood wave to be transmitted the 966 miles between Pittsburg, Pa., and Cairo. On March 22 the Pittsburg gage read 5.3 ft., which is produced by a flow in the Ohio River at that locality of about 15,000 sec.-ft. In 10 days a flood travels the 858 miles between St. Paul, Minn., and Cairo. On March 2 the reading of the St. Paul gage was 0.5 ft., corresponding to a discharge of the Mississippi of about 2,500 sec.-ft. In 8 days the effect of a flood at St. Joseph, Mo., is felt at Cairo. On March 25 the gage at St. Joseph read minus 0.1 ft., representing a discharge of the Missouri River of about 17,000 sec.-ft. If a system of reservoirs had been constructed which would have prevented all flow from the Allegheny, the Monongahela, the Mississippi above St. Paul, and the Missouri above St. Joseph, it would have reduced the 2,000,000 sec.-ft. discharged by the Mississippi River at Cairo on April 2 less than 35,000 sec.-ft.
>
> The water which passed Cairo on April 2 came principally from the White and Wabash and the lower tributaries of the Ohio, and after the water of these rivers started to subside the flood from Cincinnati, though increasing from 57 to 69 ft. on the gage, could increase flood heights at Cairo less than 1 ft. The flood of 30 ft. at Pittsburg on March 28 produced its effect on the Cairo gage on April 8. It has prolonged the flood without increasing its height.

If reservoirs are to be located on the headwaters of the tributaries, then sufficient capacity must be provided on each of the tributaries to allow for its maximum flood. This capacity will be many times the capacity estimated from the discharge of the main stream, because floods are generally caused by the flow from a number of tributaries, but seldom from the flow of all the tributaries at their maximum stage. Thus each individual tributary must be treated and reservoirs must be so operated that they will be empty when needed and when full they must be emptied slowly so as to prevent the piling up of water in the main stream.

A marked defect of this system is the possibility that one storm may follow another so closely that at the time of the second the reservoirs will be full and not available for storage purposes.

Besides the points mentioned above, there enters the question of costs. Reservoir sites are, in general, extremely costly. The Pittsburg Flood Commission estimated the cost of its proposed system at $21,000,000, but a Board of Engineers, U. S. Army, in reviewing the report found that land and property damages were underestimated by $13,000,000, so if their other estimates were correct, the cost would be $34,000,000. If a complete system were practicable for the Ohio River, some idea of its probable magnitude may be gained from the fact that the area of the basin of the Ohio Valley is 204,320 square miles, while the drainage area of the rivers above Pittsburgh just referred to is but 10,152 square miles, or 5 per cent of the whole. Assuming that the cost per square mile would be the same as that for the Pittsburg system, the total cost of providing reservoirs for the Ohio River would be $608,000,000. It is probable, however, that these costs would be greater, for the territory in question above Pittsburg is not nearly as valuable as in such states as Ohio, Indiana and Illinois.

The advocates of reservoirs for flood prevention recognize the enormous cost of this method of protection for a large river like the Ohio, and to offset this very pertinent objection they claim that the water stored in reserviors may be used to produce power which may be sold to the public, thereby reducing the cost of flood prevention, and may also be used to increase the low water flow of streams, thereby improving navigation.

It is true that reservoirs may be used for flood prevention and the water held therein used for power, or for increasing the low water flow of a stream, but the full capacity of a reservoir may *not* be used for all three of these purposes at the same time. Reservoirs to be used for flood prevention must retain the water during the possibility of floods, but the instant that possibility passes the water must be allowed to escape as rapidly as possible without doing damage, in order that they may be empty at the time of the next possible flood. The use of reservoirs for increasing the flow of streams at low water stages is just the reverse of this. They must be kept full as long as possible and the water only allowed to escape when it is absolutely necessary to increase the low water flow. The use of reservoirs for power, on the other hand, requires a constant flow with a constant head to obtain the best results, and the more nearly these conditions are met, the more efficient is the plant and the greater income obtainable from a given expenditure.

A little thought on this subject will convince a reasonable man, that the three uses of reservoirs proposed by their advocates are incompatible one with the other, and in my opinion if an endeavor were made to use the same reservoirs for these three purposes, it would not be long before the power interests would obtain control, and it would be practically impossible for a Government agent to empty a reservoir in anticipation of a flood when the power interests desired the reservoir to be kept full in order to provide uniform power for their plant.

In any case, it will be seen from the above that to prevent floods by the use of reservoirs will require a certain definite capacity; that if these reservoirs are to be used for any other purpose, their capacity must be vastly increased, thereby increasing the costs of the system. For these reasons it is believed that it would be extremely inadvisable for the Government to attempt to provide reservoirs for flood prevention combined with other uses incompatible with that object.

The cost of any system of reservoirs sufficiently extensive to have a radical effect on Ohio river floods will be very great, but owing to the scarcity of reliable information no estimate of cost can be made at present. Before undertaking or even advocating such a system the subject should be thoroughly studied, all necessary information collected, and estimates submitted based on knowledge rather than guesses. The Board, of which I have the honor to be a member, has devoted nearly a year to the compilation of all available information with reference to the Ohio and Erie basins, and it is remarkable to find how little information there is upon which to base authoritative computations. Practically the only maps upon which reservoir sites can be located are those of the Geological Survey, and their sheets cover only 45 per cent of the Ohio River basin, exclusive of Cumberland and Tennessee rivers. No location surveys have been made except in very few cases; rainfall and run-off records are incomplete and discharge records of floods stages of the tributaries are almost non-existent. Before the feasibility of protection by reservoirs in any particular case can be affirmed, available information must be supplemented by extensive, reliable data as to run-off, flood heights and discharges, available sites and costs.

INCREASED CHANNEL CAPACITY.

The capacity of streams may be increased by the use of levees, dredging, or auxiliary channels.

With reference to levees it should not be necessary to discourse at length. This means of flood protection has been used for many hundreds of years. It is a means available for the immediate protection of sites or areas of varying size. Large areas of the Mississippi basin are protected by levees as well as many towns on that and other rivers. New Orleans and Cairo are so protected. With this method the solution is simple, and the only important questions are to determine the size of the levees and the cost. An area may be made secure against any flood of similar height to those we have had in the past, or that may be expected in the future. The method is simple and direct and is the only method extensively used for protection in cases of large river systems. The levee system on the Mississippi has been in process of construction many years, and while it is true that breaks occur which cause vast damage, it is due to the fact that such levees have not been constructed to the height necessary for protection against the greatest floods. Where a community felt itself too poor to provide against the highest floods they have built their levees as high as their funds would allow, with the expectation and knowledge that when a higher flood came the levees would be topped. Shawneetown on the Ohio is an example of this policy. The Mississippi River Commission has elaborated a plan for levees which if carried out should make the lower Mississippi valley absolutely safe against overflow.

Dredging may be used for increasing the capacity of channels where the increase required is very moderate, but a radical increase in the capacity of a large stream may not be accomplished by this method without the expenditure of enormous sums of money. A point oftentimes forgotten is that dredged material must be placed outside of the river channel if the dredging is to do any permanent good. This requires the purchase of lands upon which such material may be deposited, and increases the cost of actual dredging very materially.

No instance is known where a radical increase by dredging of the flood-carrying capacity of a river channel has been attempted on any large stream. The low water channels in streams like the Mississippi oftentimes are dredged to facilitate navigation, but it is believed that the limited application of this method should be apparent to all thoughtful minds. In this connection I am reminded of the remark frequently heard that it would be advisable to bring the machinery from the Panama Canal to the United States to perform the necessary work of dredging the Mississippi and Ohio rivers. The machines formerly in use on the Panama Canal are adapted to the special conditions at Panama, which are very different indeed from the conditions to be met on the Ohio and Mississippi rivers. Most of the excavating plant consists of steam shovels, locomotives, flat and dump cars, unloading machines, etc. The dredges in use were built for deep draught work and are wholly unsuited to river work. In any case, the dredges will be required for maintenance work on the canal and the other machines are not suitable for river dredging, and if brought to these rivers would be extremely inefficient.

Auxiliary channels have been suggested, both paralleling long stretches of streams and also to form cut-offs. The cost of providing additional channels for a stream like the Ohio or the Mississippi, of sufficient capacity to carry off the flood waters, is absolutely prohibitive. For the protection of a particular locality short auxiliary channels or cut-offs may be used, but these have the disadvantage that while they may benefit the locality in question, by facilitating the runoff past that particular locality they pile the water up more rapidly below, thereby creating more damage at other points, as they necessarily cause steeper slopes, higher velocities, and greater erosion.

The methods of prevention of floods have been discussed at some length, and it must be realized that the subject is one of great extent and the problem in the case of large streams very difficult of solution.

PREVENTION OF FLOOD DAMAGE.

We now come to the question of the prevention of damage by floods. (The Board of which I am a member visited 66 different localities where the greatest amount of damage occurred as a result of last year's flood, and we found at almost every place visited that the damage was largely caused by artificial encroachments on the natural channels. For instance, many towns have grown up on the flood planes of the streams, occupying areas that have been overflowed from the beginning; people have entered the bottom lands and erected their structures with the knowledge that those lands were formed by silt deposited by flood waters; railroads have constructed earthen embankments across these bottoms, leaving only very narrow openings with wholly inadequate capacity for passing floods; city and county officials have built bridges with abutments projecting into the stream, with many piers of insufficient height, thus reducing the discharge area materially; individuals have dumped materials over the banks to increase the area of their property for business purposes. These structures and encroachments have reduced the capacity of the streams, have formed partial dams which raise the water above previous levels and then by the breaking of an embankment or the washing out of a bridge the water held back has rushed downstream under increased head and velocity, destroying everything in its path.) In our report just submitted to the Secretary of War, numerous quotations are made from the various reports of committees of the Flood Board who visited the cities damaged by the late floods, giving examples of the contraction of the waterways, encroachments on the channels and the resulting damage. Aside from the damage done by the inundation of property, which, while serious, was not destructive, almost all of the damage at the points visited was caused by increased velocities of current due to the backing up of the water, by embankments, bridges, and other structures, the subsequent breaking of these partial dams and the rush of the released waters under increased heads and velocities.

The measures to be taken to prevent such damage are: wherever possible, to remove structures from the flood planes and river bottoms, or to elevate such structures above possible high water; to protect by levees valuable property which it is impossible to remove; to prevent the construtcion of heavy earthen embankments across flood planes by railroads or counties; to increase the capacity of channels by removing encroachments thereon in the shape of bridges, buildings, etc., and to remove all artificial and natural filling or deposits.

The work of the national Government along this line up to the present time has been limited. The principal work has been the construction of the levees on the Mississippi river. No other extensive work has been carried on for flood protection or flood prevention alone, although many of the works for improvement of navigation have tended to improve conditions with reference to floods. At the time of the great flood of last year the question arose as to the powers and duties of the general Government, and while it is probable that the power of the Government is sufficiently extensive to enable vast works to be carried on, there are at the present time no laws authorizing such works, unless they can be combined with works of the improvement of navigation.

As is well known, the national Government derives its authority for its various activities from the provisions of the constitution of the United States. It is a principle in the interpretation of the constitution that all rights are reserved to the states except those which are specifically and clearly given to the national Government, but once it is determined that the right obtains, it is possessed to its fullest extent, is superior to state rights, and is paramount even to the minutest details. Among the powers given to Congress by the constitution is: "To regulate commerce with foreign nations and among the several states, and with the Indian tribes"; and "to establish post offices and post roads." Under the former of these provisions the Government has assumed control of navigable waterways and regulates navigation. Insofar as floods interfere with navigation there is no question as to the right of Congress to pass laws for the control of such floods or the prevention of damage which may affect navigation. It matters not whether the laws have for their purpose the lowering of flood heights or the prevention of the wrecking of bridges, boats, etc., or the deposit of material washed from the upper parts of the stream into the lower and navigable portions. This power has been exercised in the past through the engineer department of the Army in charge of river and harbor districts, but has been limited in general to those stretches of the streams that are actually navigable. At the present time we are regulating the heights and widths of bridges, preventing deposit of material over the banks of the streams, or placing obstructions in the stream, etc., and on special occasions where it has been shown that material deposited in the stream above the navigable sections has been washed downstream, affecting navigation, the Federal Government has interfered and prevented such deposit. In general, however, its jurisdiction has been confined to the navigable portions of the streams. It would seem perfectly possible and within the powers of Congress derived from the constitution to authorize the extension of jurisdiction to all portions of navigable streams whether actually navigable or not.

Furthermore, Congress in its power to establish post offices and post roads must naturally have the power to protect such roads, and to make regulations regarding them. As almost all the roads are used for carrying the mails, apparently Congress has the power to control the construction of bridges over nearly all streams, and also to prevent encroachments thereon, on the grounds that such roads are used as post routes and must be so constructed and so protected as to be without danger of obstruction or interruption due to floods.

It would seem, therefore, that Congress has in the two provisions of the constitution quoted above, ample power to control these matters and to take such action as may be deemed wise for such control. At the present time, however, the executive departments whose powers are definitely determined by law, have no control over streams unless they are navigable, and no control over the action of individuals, corporations or communities to prevent the obstruction of such streams, unless it can be definitely shown that such obstructions do interfere with navigation; and as soon as one of the departments attempts to exercise its power, unless the case is simple, objection is made and suit entered in the courts to determine whether under the laws that department has the right to demand correction or not.

It would seem that the measure most needed at the present time is an act definitely authorizing some department of the Government to prevent the encroachments on the streams, and if the Federal Government does not take up this question, it is within the power of the states to do so, and some measure or measures should be taken in each of the states to enable some department of their Government to control such matters.

There are numerous cases where the people of the community have recognized the great danger involved in the construction of a bridge for instance, and where protests have been made but without avail, because of the lack of laws under which any authority could act with hope of successful enforcement. It is a remarkable fact that in one of the towns of Indiana certain county commissioners are constructing bridges across a stream in direct opposition to a large proportion of the citizens of the town, the latter feeling confident that the bridges in question will cause great damage in the case of another flood, yet these citizens have been unable to find any power that would enable them to prevent the construction of such bridges.

If the Congress of the United States should take up this matter and should extend its jurisdiction to the headwaters of all tributaries of navigable streams, it would be possible for the agents of the Federal Government to determine whether or not proposed structures are dangerous to life and property during floods and to require reasonable and safe construction. In the meantime the Board of Officers, of which I am a member, is studying all conditions affecting flood protection of the Ohio River basin, and has collected practically all of the information available. It has recently recommended in its report that an appropriation be made for carrying on this work and for making surveys and plans for definite improvements of localities and streams.

The Board feels that the problem of prevention of floods in the Ohio River basin is an immense one, whose difficulties are little understood by the mass of the people. A great deal of information is being disseminated, whose most remarkable quality is the lack of substantiation by definite facts. Newspaper and magazine articles are describing and advocating measures without knowledge of what they will ultimately mean to the people and we feel that the first duty of the Board is to obtain definite and precise information on all these subjects, so that the people may know what can be done and what it will cost. We are not opposed to any method that will give relief and as I have shown in the above discussion there are many methods and measures that may be of assistance. It is probable that when the matter is sufficiently studied it will be found that all of the methods described above may be used to advantage, and that each locality and each stream is a problem by itself, whose solution depends on local conditions and can only be determined when adequate knowledge is secured.

The Western Railway of Buenos Ayres will eliminate all existing grade crossings and will electrify its suburban zone for a distance of about 25 miles from the terminal, according to a recent report.

Railways

A Pneumatic Concrete Mixing and Conveying Plant Mounted on a Railroad Car for Tunnel Lining.

(Staff Article.)

A new type of portable concrete mixing plant will be used by the Chicago, Burlington & Quincy Railroad for lining the Alkali Summit Tunnel near Arminto, Wyo., a station 19 miles west of Powder River on the Sheridan Division recently constructed between Powder River and Thermopolis, Wyo. The tunnel is on a grade and has a length of about 1,000 ft. On account of the nature of the rock strata through which the tunnel penetrates it has been decided to line the tunnel with concrete.

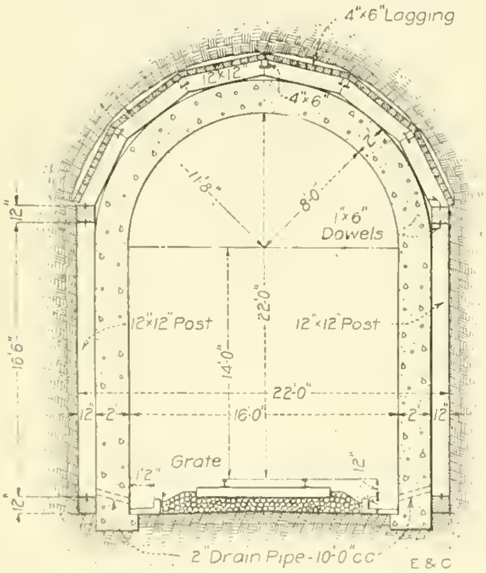


Fig. 1. Cross-Section Showing Concrete Lining and Track Gutters to Be Constructed in Alkali Summit Tunnel, C. B. & Q. R. R.

The cross section of the tunnel and lining, as shown by Fig. 1, is the standard single-track tunnel section of the C., B. & Q. R. R. used where timbering cannot be removed, with the exception of the concrete gutters or ditches parallel to the track to be constructed in this case on account of the water flowing in one direction through the entire length of the tunnel. Cast iron grates will be placed every 10 ft. along the track side of each gutter to allow the water to flow from the track and ballast into the gutters.

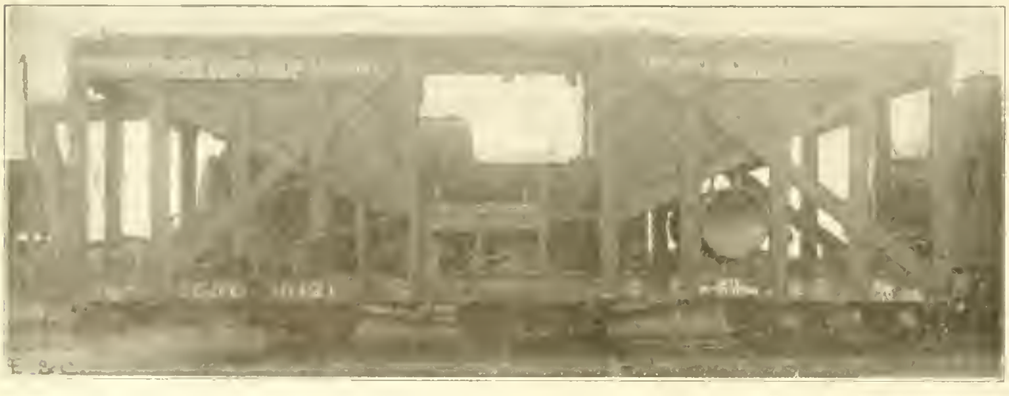


Fig. 2. Side View of Pneumatic Concrete Mixing and Conveying Plant, C. B & Q. R. R.

The pneumatic mixer and conveyor to be used on this work is mounted on a standard 40-ft. flat car, as shown by Fig. 2. The bins erected on the car contain 705 cu. ft. of space for the aggregate, which will consist of bank run gravel. These bins slope to the center of the car and discharge through chutes into a measuring hopper. The cement is stored in bags under one of the bins. This bin is 18 ins. narrower than the car, which leaves space for a man to carry cement bags from the bins up a slight incline to the mixer platform. A description of the pneumatic concrete mixer and conveyor and of its use for lining the La Salle St. tunnel in Chicago was given in Engineering and Contracting, June 14, 1911.

The measuring hopper is lifted and tilted to discharge into the mixer by means of a cable operated by a small air cylinder. The pipe which conveys the concrete from the mixer to the tunnel forms is 8 ins. in diameter and runs from the mixer under the car and up the end to the top of the bins, which are 14 ft. above the rail, and then on an incline above the car to a point about 24 ft. above the rail, where it enters through a bulkhead of the tunnel form. This incline pipe is raised and lowered in order to clear the braces of the tunnel forms which only give about 16 ft. of clearance. This pipe is raised and lowered between guides by a cable attached to an air cylinder. When the pipe is lowered to its resting place on top of the bins the guides fold down and are out of the way. Figure 3 shows an end view of the car and the conveying pipe folded down out of the way so that the car can be moved. The incline up which the cement is carried is also shown.

The water barrel is located on the car and the water is discharged from it through a pipe to the mixer and the barrel is supplied by a small hose connecting with a pipe line along the floor of the tunnel.

The car is run into the tunnel with enough material for about 25 cu. yds. of concrete and is connected to an air supply line laid along the track at whatever point the work is to be carried on, and may be unloaded in one hour, mixing, conveying and placing 25 cu. yds. of concrete in the tunnel forms with a crew of four men. The crew required is as follows: One operator who operates the mixer and all the valves; one man to carry cement; one man to untie the cement bags and dump the cement into the hopper, and one man to operate the gates for filling the measuring hopper with aggregate. The air pressure used to operate the mixer and conveyor is 80 lbs. per square inch. The concrete lining will be placed in sections having a length of from 5 to 10 ft. along the axis of the tunnel. The sequence of concrete placing operations for each 5 to 10 ft. section is as follows: One side wall will be built up to a height of 5 ft.; then the other side wall will be brought up to the same height; next the first wall will be brought up to the 10-ft. level. This alternate building of the side walls continues until concreting has reached points well up onto the haunches of the tunnel arch. Finally the remaining top section of the arch is placed.

The work will be done by railway company forces. Mr. T. E. Calvert is Chief Engineer of the C., B. & Q. R. R. The concrete mixers and conveyors are manufactured by the Concrete Mixing & Placing Co., Chicago, Ill

Principles and Methods for the Valuation of Railroad Property and Other Public Utilities.

A committee of the American Society of Civil Engineers was appointed in September, 1911, to formulate principles and methods for the valuation of railway property and other public utilities. This committee consisted of Frederic P. Stearns, Leonard Metcalf, Thomas H. Johnson, Alfred Noble, William G. Raymond and J. P. Snow. At the annual meeting held in January the committee presented the report on Valuation for the Purpose of Rate-Making. This report slightly condensed follows:

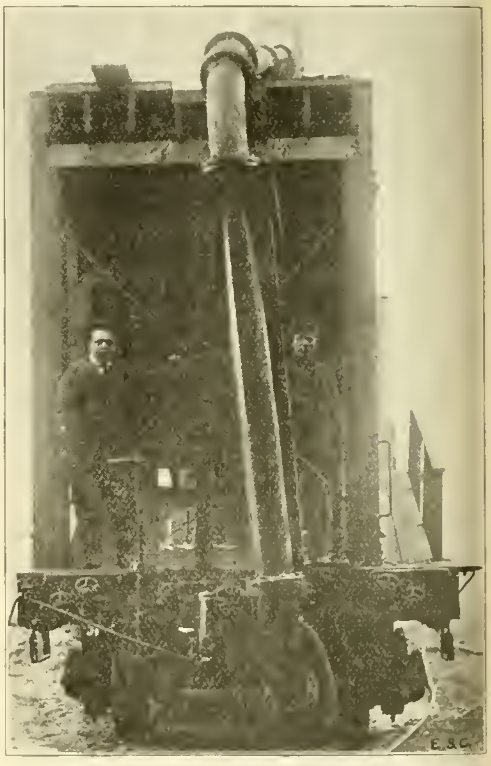


Fig. 3. End View of Pneumatic Plant for Tunnel Concrete Work, Showing Conveyor Pipe and Incline to Mixer from Cement Storage Under Bins.

Valuations of the property of public service corporations may be made for a variety of purposes, the most important of which are the furnishing of a basis for the regulation of rates and for taxation, capitalization and purchase.

It is sometimes held that valuations for all of these purposes should give identical results. This view cannot prevail if the laws of the different states and certain decisions of the courts are complied with. For instance, there are in many states laws indicating how the property of public service corporations shall be valued for taxation and these laws differ radically in their requirements. There are other reasons why the valuation for different purposes should differ, but this feature will not be discussed at present as this report will, as stated, relate to valuation for the purpose of rate-making.

It is, of course, true that the principles and methods of valuation for rate-making are in most respects the same as those involved in the valuation for other purposes.

The valuation of public utilities has attained special importance in recent years and most of the literature relating to this subject has been issued within the last thirteen years. The principles and methods involved are now being carefully considered by many public service commissions and the courts, as well as by engineers, lawyers, and committees of various societies having to do with public utilities. The subject is clearly in the developmental stage, and any conclusions reached at the present time must be considered as tentative.

Until recently there were few, if any, engineering or legal works devoted exclusively to the valuation of public service properties, but during the year 1912 and since, several extended and valuable works upon this subject have issued from the press. Especially noteworthy is a comprehensive work of Mr. Robert H. Whitten*, a book of nearly 800 pages, which contains a well arranged, clear and concise discussion of the various principles involved in the valuation of the property of public service corporations and is replete with extended quotations from the opinions of the courts. Your committee has been greatly aided in its work by Mr. Whitten's contribution to the solution of this difficult problem.

It is obvious that a valuation for use before a court or public service commission must, in order to be sustained, be made in conformity with existing laws and the decisions of the higher courts. It is recognized, however, that the whole subject is in a developmental stage, and while your committee has been guided largely by the decisions of the higher courts and public service commissions, it recommends what seem to be sound views and desirable changes in practice, even though not wholly in accord with such decisions.

GENERAL PRINCIPLES INVOLVED IN VALUATION FOR RATE-MAKING.

The primary requirement in the valuation of a public service property is that justice shall be done both to the owner of the property and to the public. The valuation is for the purpose of determining rates which shall be a limitation of charges to those which will give a fair return and no more. In some degree the power assumed by the public to limit rates implies a guaranty, in the case of a properly managed normal property, of such fair return, and an assurance to the public of satisfactory service in consequence of such implied guaranty. The exercise of the power to regulate rates should be based on equity to both parties.

It has been well settled by the highest courts that the owner of such property is entitled to a fair return upon a fair value of the property utilized in or reasonably necessary to the service.

These general principles do not enlighten one as to how the fair value and fair return are to be determined, but they indicate a purpose, and it is often helpful to test any proposed method of determining value by the application of these principles

Justice and equity require that in any regulation of rates fixing the amount which may be earned by a public service corporation there shall be taken into consideration two distinct features.

First, the annual return covering interest and profit, to which the corporation is entitled for the use of its capital, having in view the risks incidental to the investment.

Second, an allowance sufficient to provide for the net depreciation in the value of all the items of physical property, whether resulting from decay, wear and tear, or other cause, the amount of such depreciation allowance to be sufficient to amortize all such items of property by the time they cease to have value

The principles are similar to those involved where one man loans money to another. It is not enough in such a case that the borrower should pay interest on the money loaned, but he must also pay the principal to cancel his indebtedness.

In many instances there has been a failure to recognize this principle in the case of public service properties by reason of the consideration of the property as a whole and not as the sum of its parts, and the situation becomes much clearer when one considers separately each item of property having a limited life. Each such item may be said to have a value when new equal to the capital invested in it, and by the time such item ceases to have value, the corporation is entitled to have received from the public the full amount of its capital; at least this is the general principle involved.

It is obvious that a fair return must include not only the items above mentioned, but must include sums sufficient to cover operating expenses, taxes, and all other current expenses, and such portion of the cost of maintaining the property in good condition and of replacing portions of the property as are not covered under the head of the depreciation allowance. The line of separation between maintenance and replacements, on the one hand, and the depreciation allowance on the other, will be discussed later in one of the sections under the head of Depreciation.

Public policy requires that, in the absence of any plan for public ownership of public service property, the earning capacity of such properties be made sufficiently attractive to induce capital to provide the works needed for the public welfare and convenience, to maintain them in proper condition for providing good service and to extend them reasonably in advance of the needs of the community. The public may be as injuriously affected by too low as by too high rates.

If the public regulation of rates to be charged by public service corporations is to become general, the fairness and uniformity of methods of valuation for rate-making will have an important influence upon the supply of capital for such enterprise, and upon their maintenance and development so that they will furnish satisfactory service to the public.

FAIR VALUE FOR RATE-MAKING.

As already indicated, no complete and definite rules have been laid down for the determination of the fair value of a public service property. The matter is so difficult that courts and commissions have in the majority of cases stated that they have given weight to valuations determined by several different methods, and that, after due consideration, have reached a certain conclusion which they announce without disclosing definitely the method by which the conclusion has been reached. This appears to have been a wise course to pursue during the developmental stage of this subject, in view of the weight as a precedent given to important court decisions.

In all private transactions the controlling feature in the valuation of a property which has a capacity for earning is the probable future earning capacity, and this, while considered from many points of view, is determined mainly from the financial history and present physical condition of the plant. The value obtained in this way is substantially what would be the market value of the property were there conditions creating a market, and is the amount which a willing and intelligent buyer desiring such property would give for it to a willing and intelligent seller

During the former periods when rate regulation was rarely enforced, the value of the public service properties was determined in substantially the same way as the value of private property, and at the present time the stocks and bonds of such properties are valued to a large extent on the basis of their earning capacities

As the future earning capacities of such properties under rate regulation by commissions will to a large extent depend upon rates to be established as a result of valuations of the properties, it is clearly illogical to base the valuations on present or past rates. Such valuation would be reasoning in a circle. The value used for rate making should be determined in such a way as to lead to equity as between the owner, on the one hand, and the consumer or the public on the other

There are some cases where, on account of high cost of construction and operation, competition, or other reasons, public service properties will not furnish an adequate return on the investment whatever rates may be fixed. In such abnormal cases there cannot be what is usually regarded as a fair return upon what would under ordinary conditions be considered the fair value of the property. Such a property may be classed as a losing venture, to which many of the principles and methods applicable to normal properties will not apply.

METHODS OF DETERMINING PHYSICAL VALUE.

Two methods are in use and have the qualified approval of the courts; the first of these is based on the actual cost of the work, with suitable provisions for the cost of developing the business in the earlier and unprofitable years of its life, and with deductions for depreciation, etc.; the second method consists of an estimate of the cost of reproducing the property with similar adjustments.

In the case of a property recently created, with accounts kept properly, the best basis for a valuation is the actual reasonable cost as shown by the accounts; in such case the difference between the two methods is nominal, because the best data for an estimate of cost of reproduction will be furnished by the accounts; but it will be necessary to scrutinize the accounts and to examine the property to ascertain whether the construction was carried out in a judicious manner. The criterion should be whether the cost was reasonable. The public should not be required to pay a return upon extravagant or injudicious expenditures; on the other hand, the rates should secure to the owner a fair return on his reasonable outlay.

In the case of an old property, it is generally impracticable to ascertain what the reasonable cost was. If the accounts can be found at all they are likely not to represent fully the cost of the property, owing to the failure of such accounts to properly separate charges for the construction of new work from those for operation and maintenance. Moreover, if the books include only cash payments, they may exclude the value of interest on capital and the value of important services where such services were not paid for in cash.

The difficulty of determining the value of an old property from the accounts is greatly augmented by the fact that the cost has been continuous from the first construction to the present time, and that portions of the cost are being extinguished each year by earnings offsetting the depreciation of the perishable property. The cost at the time the work was first put in service may have little relation to present value or to the present investment. This can best be shown by an illustration:

Take the case of a railroad fifty years old, built at first as a cheaply constructed single-track line, including as a part of the original property cheap land, light iron rails, wooden bridges, wooden trestles over low ground, small stations of wood, and light-weight locomotives. In the course of fifty years some of the land has increased greatly in value, other parts have been abandoned for railroad purposes owing to the relocation of the railroad with easier curves; the iron rails have been replaced from time to time with heavier iron rails, and subsequently with still heavier steel rails; the wooden bridges have been replaced with iron and steel bridges; the trestles have been replaced by solid embankments; the roadbed has been widened from time to time, first in places for the building of turn-outs or side-tracks, and later continuously for the laying of one or more additional main tracks; the station buildings have been enlarged and subsequently replaced with modern structures of stone or brick, and the locomotives have been replaced from time to time by heavier and more powerful machines.

It is easy to see that under such circumstances the actual cost of the original railroad has very little relation to the value of the corresponding part of the present railroad. The present value on the basis of actual cost should, of course, include all additions, but when one considers the manner in which such additions are made it will be seen that wonderfully good bookkeeping would be required to supply intelligible and reliable data for a complete ascertainment of cost. For instance, rails of greater weight have been put into use as they were replaced from time to time, those now in the track weighing about twice as much as the original rails. What railroad manager or accountant could have been expected in past years to discriminate between replacements and betterments so closely as to make the accounts represent the added value of the property due to such changes with a reasonable degree of accuracy? In addition to these difficulties, there is the further one of writing off correctly the value of those items of property which, instead of being replaced, have become obsolescent.

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*"Valuation of Public Service Corporations," by Robert H. Whitten, [illegible] New York, 191[illegible]

The result obtained after making deductions for obsolescence would be the value of the existing items of property new, which should be further diminished by the net amount of the depreciation of the existing property.

The total amount which would have to be deducted for obsolescence and depreciation would be better appreciated if one, instead of dealing with the method of accounting above indicated, in which the depreciation of property is partly offset by charging replacements to operation had a set of books showing the cost of construction and equipment, including every item of property built or purchased, whether the original item or a replacement.

If the property under consideration were a railroad fifty years old, there would be a very few items of the original plant that would not have gone out of use, so that if the cost of all obsolete items were written off, including all overhead charges appertaining directly thereto, there would remain only the cost of a few long-lived items and the other existing items of property to be considered.

Attention is called to this phase of the question because, if the assumption is correct that the total cost of all obsolete items should be deducted, there remains to be considered only the actual cost of the existing items of property. This method, too, would eliminate the uncertainties inherent in accounts which attempt to charge replacements to operating expenses and maintain the cost of the original items of property on the books as charges against capital.

If the value of a very old property, made up of long-lived items, were ascertained from the accounts, it would contain the feature that similar parts of the property would be valued at widely different rates in accordance with prices at the times when furnished. For instance, in an existing water-works property the pipes laid 45 years ago cost $65 a ton, those laid 15 years ago, $18 a ton, and those laid during the past year, $24 a ton. Such a divergence in the estimated value of essentially identical items of property, although apparently anomalous, is not necessarily inequitable in a valuation for rate-making purposes.

For reasons given, the committee believes a valuation of an old property based on its actual cost to be generally impracticable, making it necessary to adopt the cost-of-reproduction method. Many questions remain to be settled in the application of this method which cannot be treated in a general statement and will be discussed at length in subsequent portions of the report under the heads of Identical versus Substitute Plant, Unused Property, Present or Original Conditions, Unit Prices, Overhead Charges, Development Expenses, Depreciation and Appreciation.

ACTUAL COST METHOD OF VALUATION FOR FUTURE PUBLIC SERVICE PROPERTIES.

In the opinion of the committee it would be entirely just and equitable, and it is highly desirable to provide by law that future public service properties should be valued on the basis of their actual reasonable existing investment, and to determine or limit rates upon such a valuation if the service rendered permits.

In this case the corporation would and should be relieved from the risk of gain or loss due to the rise or fall of prices of labor and materials, which is sound public policy, since to reduce the risk of investment is to lower the rate at which money can be obtained. A valuation based upon book accounts can be kept up to date by adding to the original valuation the cost of additions and replacements made from time to time to the property, and by deducting the amounts which the corporation is permitted to earn annually to offset the net depreciation of the property.

By this method the considerable expense involved in the revaluation of the property from time to time by the reproduction method would be avoided, and the value obtained would not fluctuate with the variation in the prices of labor and materials. It would also be feasible through changes in the depreciation allowance to amortize abnormal but legitimate costs and to keep the book value of the property reasonably commensurate with its cost of reproduction value, even in the face of rising or falling prices or permanent change in conditions.

IDENTICAL VERSUS SUBSTITUTE PLANT.

In applying the cost-of-reproduction-less-depreciation method, the question has sometimes been raised as to whether the property to be reproduced is substantially the identical existing property or a substitute hypothetical property of modern design, capable of producing the same results as the plant to be valued.

In considering this question from the standpoint of equity to both the corporation and the rate-payer, it is necessary to take into account the amount which the corporation should be permitted to earn annually for depreciation.

If, in computing the annual allowance for depreciation, account is taken only of the lessening of worth of the various items of property due to physical decay approaching inadequacy or obsolescence, it would not be fair to the corporation to further depreciate the value of its property because some new design of works would produce the results obtained by the old plant at a smaller cost.

To illustrate, a water company may have laid main pipes from time to time from its sources of supply to a city, until it has three. In such a case one or two pipes having the same aggregate capacity as the three might be laid at a considerably smaller cost, but it would be unfair to the corporation to diminish the value of its pipes for this reason, unless it had been permitted in previous years to earn an annual sum for depreciation sufficient to cover the lessening of worth due to improvement in design in addition to the sum required to cover physical decay, inadequacy and obsolescence. (See Depreciation.)

Practically it is not good policy to provide this further depreciation allowance or to adopt this theory of value. It is desirable to adopt a uniform method, and there are many reasons why the method of valuing the substantially identical property is preferred.

First, the weight of court decisions favors this method, although no decision has been given by the United States Supreme Court.

Second, it is much less difficult to determine the reproduction cost of an identical plant than of a substitute plant which will produce equal results. Different engineers would undoubtedly differ greatly in their views as to the location, design and cost of a substitute plant, so that the results might be regarded as speculative and the cost of making the valuation would be largely increased.

Third, allowances for depreciation in the past have generally been too small to make it just to apply the substitutional method to existing properties.

Fourth, the adoption of this theory would substantially increase the depreciation requirements and the charges to be met in the early stages of plant development.

UNUSED PROPERTY.

It is frequently the case, especially in connection with water works properties, that land is acquired with reference to its future use, as for instance for storage or distributing reservoirs, or that water rights are bought to meet future needs; and in nearly all old plants there is property which has either been wholly discarded or is maintained for use in cases of emergency. The courts have generally in theory, and sometimes in practice, excluded property which is unused, and in some cases parts of that used on the ground of unnecessary size or capacity.

The following language has been used in the decisions of the United States Supreme Court referring to the property to be valued:

> Value of property used * * * for the convenience of the public.
>
> Value of the property actually used for the purpose of supplying water.
>
> The fair value of its property devoted to the public use.

United States District Judge Farrington, in a San Francisco water rate case, in 1911, ruled that:

> Only that property is to be considered which was then used and useful in supplying * * * water.

In view of these decisions, an engineer in ascertaining a value for rate-making must take into account whether or not the property, although owned by the corporation, is in use. He should also consider whether portions of the plant are unnecessarily large, as a New Jersey Chancery Court in 1905 deducted nearly $130,000 from a total cost of $1,400,000, in view of the fact that certain portions of the plant were larger and more expensive than was reasonably required. A part of the deduction was made because a 36-in. main was used where a 30-in. main would have been sufficient, and because a dam was made of excessive width for present purposes in order that it might be subsequently raised.

Judge Savage of the Maine Supreme Court used this illustration:*

> Suppose that a five-hundred horse-power engine was used for pumping when a one-hundred horse-power engine would do as well. As property to be fairly valued, the larger engine might be more valuable than the smaller one, yet it could not be said that it would be reasonable to compel the public to pay rates based upon the value of the unnecessarily expensive engine.

It seems rather unfair and against good policy that a corporation wisely purchasing, for example, land which has special fitness for its purposes while it is available at a price far below its probable future value, should be required to hold such property without any return on the investment, and to make such returns would be entirely reasonable and fair to both parties, provided the net income of such land, including its appreciation in value, were credited to the public. On the other hand, it is obvious that a ruling of the court which permitted the corporation a return on all land purchased might lead to abuses, as the corporation might purchase land for speculative purposes and subsequently dispose of it, so that the community, although paying a return upon its value for a time, might receive no benefit from such land.

While the rulings of the court in regard to such unused property are not unjust, because the corporation may defer its purchase unless it is willing to take the risk that the advance in value will furnish sufficient compensation for holding the property, it is very desirable, in some cases, in the interests of economy, that the corporation should anticipate future needs. For instance, by purchasing a site on the top of a hill, where a distributing reservoir may be placed, before such a hill property is subdivided into streets and house lots, and by acquiring water rights when they can be obtained at small cost, lest it may later have to pay substantially monopoly prices. This last instance is particularly applicable to the semi-arid and arid regions in the western and southwestern United States.

It is clear that if a corporation does purchase land and water rights before they are required for use, and they are excluded from valuation as a basis for rate determination, later, when they do come into use, their cost should not be the criterion of value but the then value of the property.

While a community should not be required to pay rates upon the full valuation of a plant, which through negligence or very bad judgment has been built of an unreasonably large size, care should be taken in the application of this rule not to omit any portion of the plant on account of large size, if ordinary and reasonable judgment has been used in the designing of the works. Ultimate economy and the convenience of the public require, for instance, that the main water pipes in the streets be made adequate for a considerable period in the future. On the same grounds, a pumping station may wisely be made large enough for the additional pumping engine which will be required in the future, or the lower part of a dam may properly be built with reference to a subsequent increase in height of the dam.

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*Brunswick and Topsham Water District v. Maine Water Co., 99 Maine, Dec. 14, 1904.

The public service corporation should not be subjected to the hazard of having a part of the value of its plant omitted on account of large size, unless it has been clearly negligent; and the opinion of intelligent engineers as to the provision which should be made for the future differs materially, therefore deductions for the reasons stated should be made only in extreme cases, if at all.

The opinions of the courts contain discussions upon the exclusion of unusual property, or that in excess of present needs, which restrict the amount of such property to be valued to a greater extent than seems to the committee to be desirable.

PRESENT OR ORIGINAL CONDITIONS.

In the application of the cost-of-reproduction-less-depreciation method a very important question arises as to whether the reproduction cost should be based on the present conditions or on those existing at the time the works were built. A very similar question arises with regard to the valuation of land, whether present or original conditions should be used.

The committee believes it to be a general basic principle that where money has been expended with due discretion for a public service property, the company is, in the case of a newly created normal property, entitled to a fair return upon a value which, subject to modifications, equals the amount of money invested.

As the same property becomes older some portions depreciate in value as the result of decay and wear and tear, and others appreciate; there may also be a further depreciation or appreciation due to the fall or rise in the prices of labor and materials, and some parts of the property may cease to exist.

Except for these changes, which are often of great importance, and similar changes, all of which result from age, there is little if any difference between the physical valuation of an old and a new property, and the reasonable investment in one should be protected as well as in the other.

In view of these and other considerations, the committee is of the opinion that valuations should be based upon the physical conditions existing at the time the various portions of the property were built, but at the prices prevailing at or near the time of the valuation. The discussion of the subject may best be approached by taking specific cases.

Take first the Wachusett Reservoir, constructed to supply water to the Metropolitan District of Massachusetts. In constructing this reservoir it was necessary to acquire land upon which were many buildings and mills with their water power. Included in the reservoir site, also, were many highways and two railroads. As a substitute for the highways obliterated, others had to be built around the margins of the reservoir, and some were raised above the water level.

One of the railroads was relocated for many miles at one side of the reservoir and another was raised. To make the reservoir a better receptacle for water, the surface soil was stripped from its whole area. One main dam and two subsidiary dams were necessary for holding the water. Incidentally, under special laws, damages had to be paid for real estate which was not acquired but which was said to be indirectly damaged by the construction of the reservoir.

How should such a reservoir be valued on the basis of the cost of reproduction?

As a newly created reservoir, there seems to be no question but that its value should be substantially its reasonable cost, including engineering, legal expenses, interest during the construction period, and all other overhead charges and incidental expenses. An estimated cost of reproduction should produce the same result except as the value would be modified by changes in prices and by the effects of age.

At present there are in the reservoir no buildings, mills, water powers, highways or railroads except the highways and railroads which were raised.

The title to the highways and the new railroad outside the reservoir vests in the adjacent municipalities and in the railroad company, so that they would not appear in a schedule of reservoir property.

The only way to do justice in such a case is apparently to estimate the cost of reproducing the reservoir on the basis of the original physical conditions, that is, with a knowledge of the buildings, mills, and water powers destroyed and the new roads and railroads necessarily built to take the place of those obliterated.

In other words, due allowance should be made in estimating the cost of reproducing such a reservoir for every reasonable expenditure required by the conditions under which the work was done.

For other examples, consider the great terminal works recently built in New York for the New York Central and Pennsylvania railroads.

The New York Central terminal has been constructed in a piecemeal manner so as not to interfere with the operation of the trains, and consequently has cost much more than it otherwise would.

The Pennsylvania terminal has been constructed without much interference with the work.

It would obviously be necessary to take into account the different conditions under which the work was done in these two cases in order that the respective roads might have a fair return on money reasonably expended.

In some cases it may be difficult, if not impossible, to ascertain the original physical conditions, although such information should be available except in the case of very old properties. Where such conditions are not clearly known, the person making the valuation should, on the basis of such information as is available, determine as well as he can the probable conditions. For instance, it would be inequitable to assume that a large reservoir site in a settled district had no highways passing through it which required relocation and no buildings upon these highways, or to assume that the land taken for a terminal in a large city did not have buildings upon it. Equity requires that no real element of value shall be omitted because it is not capable of precise determination.

If the committee's view that the estimated cost of reproducing a property should be based upon the original physical conditions is correct many controverted features of the cost-of-reproduction method will be settled and other features will be established.

For instance, the cost of laying a water pipe in a street with an asphalt pavement is much greater than in an unpaved street, and there has been much discussion as to whether, in a case where the pipe was laid before the pavement, the reproduction cost should include the cost of taking up and replacing the pavement. The adoption of the view that original physical conditions govern, necessarily excludes the cost of the pavement.

If after a pipe has been laid in a street new grades were adopted, the reproduction cost of the pipe should be estimated on the basis of the original grades, and if the regrading required a raising or lowering of the pipe already laid, the cost of such work, if not charged against the rate payers as a part of the current expenses, should be included in the reproduction cost.

It is frequently the case that the greater part of a water works pipe system is laid from year to year as the population increases and the streets are extended. Where this has been the case, the greater cost of piecemeal construction should be included in the valuation to the extent that the method has been used.

It will also be seen from what has been stated that, if the original conditions govern, it is necessary in valuing a property to consider the required expenditures irrespective of the title to the property. One example is that already given where the committee has held that the cost of relocating highways and railroads, necessitated by the construction of a reservoir, should be included in the valuation of the reservoir even though the title to these properties does not vest in the owner of the reservoir. Another example is furnished in the case of gas or water works systems where the consumers are required to pay the cost of the service pipes leading from the mains to their property. These service pipes are owned by the company, but it would be unfair to include them in the valuation and require the rate-payers to pay rates on property for which they had furnished the capital.

On the same grounds, a street railway company which has been required to lay pavements in connection with its track-laying in the streets should have such pavement included in the inventory when a valuation is made.

In the construction of works required for the separation of highway and railroad grades, the cost is sometimes paid by the railroad and in other cases it is divided between the railroad and the municipality and state. In all such cases the original conditions under which the separation of grades was accomplished should be considered and the valuation based upon the property created at the expense of the corporation and not upon the present ownership of the property.

In determining what are often called the overhead charges, including preliminary expenses, incidental expenses during construction, and development expenses, the committee believes that the original conditions should control, as in the case of other items. The adoption of the view that original rather than present conditions should control in determining these charges will have an important bearing upon their amount, because the expense of promoting and developing the original plant is generally a much larger percentage of the total cost than of promoting and developing the additions and extensions made from time to time. The cost of engineering and administration for additions may be as large as for the original plant.

Although the committee believes that in such cases as those cited the property to be valued is that which the corporation has paid for rather than that to which it holds the title, there are obviously many cases in which property donated to the corporation should be included in the valuation. As an instance of this kind, land and money may have been donated for the construction of a railway in a location where otherwise it could not have been constructed or would have resulted for a long period in a losing venture.

APPLICATION OF REPRODUCTION METHOD OF VALUATION

For making a valuation by the cost-of-reproduction-less-depreciation method, the engineer first obtains or prepares an inventory (see Inventory) of the several items making up the physical property devoted to the public use (see Unused Property). To each of these he affixes a price or estimated cost based upon prices prevailing at or near the time of valuation (see Unit Prices). These prices whether based upon the experience of the engineer or upon prices furnished by manufacturers or contractors, do not cover the cost of promoting, organizing and financing a public service property (see Preliminary Expenses), nor the cost of engineering, administration, contingencies, and interest during construction (see Incidental Expenses During Construction), and it is therefore necessary to add sums or percentages to cover these items.

During the early years of the operation of most public service properties a deficiency of income accumulates, which should be included in the valuation (see Development Expenses), and working capital is at all times needed for the operation of the property, which should also be so included (see Working Capital).

While many of these costs are sometimes classed, erroneously, as intangible, they are in fact actual and necessary expenditures for the creation of the physical property, and should therefore be included the same as other necessary and proper expenses.

In order to complete the valuation by this method, it is necessary to make a deduction from the values obtained, as above stated, for the net accrued depreciation of the physical property due to age and other causes (see Depreciation).

After such deduction has been made, the result is the value of the physical property by the cost-of-reproduction-less-depreciation method. Whether and how far it is or is not a reasonable measure of the fair value of the property upon which a fair return should be made will be considered later (see Should Rates be Based on Physical Value Only?)

INVENTORY.

The inventory should be made in much detail and with much care in order to obtain the best results. Even under such circumstances the quantities of material will generally be too small unless those making the inventory recognize that, in practice, the actual quantities nearly always over-run the theoretical or geometrical quantities. Take, for an extreme instance, the dredged channel of the Panama Canal leading from the deep water on the Atlantic side to a short distance beyond the shore line. The Annual Report of the Isthmian Canal Commission to June 30, 1912, states that the geometrical quantity represented by the excavation of this channel to that date was 10,169,000 cu. yds., while the actual quantity taken out of the channel amounted to 22,886,000 cu. yds., an excess of 125 per cent. The excess in this case was due to the quantity of material washed into the channel by waves and currents.

In the building of an earthen dam across a river of considerable size, it is the rule, rather than the exception, that floods occur which damage coffer-dams and wash away considerable material. If the prices are predicated upon the quantity of earth excavated to make the dam, due allowance should be made for the material washed away or otherwise unavoidably wasted.

Under the specifications usually adopted, the actual weight of cast-iron water pipes and much other metal work is in excess of the geometrical quantity, and due allowance should be made for such overweight.

Where contracts are made at a given price per cubic yard, it is frequently necessary during the progress of the work to make changes which require the taking down and rebuilding of certain parts of the work, and this increases the quantity of work for which the contractor is paid beyond the quantity actually in the final structure.

It is also the case that in the valuation of many public service properties it is practically impossible, within the limits of reasonable cost of valuation, to make the inventory complete. Under such circumstances it has been customary and is proper to allow a small percentage, varying inversely with the completeness of the inventory, for omissions.

UNIT PRICES.

The rulings of the highest courts indicate quite clearly that it is the present rather than the original prices that are to be used in determining the cost of reproduction; for instance, in the Consolidated Gas Company case,* decided in 1909, it is stated:

> that the value of the property is to be determined as of the time when the inquiry is made regarding the rates. If the property which legally enters into the consideration of the question of rates has increased in value since it was acquired, the company is entitled to the benefit of such increase. This is, at any rate, the general rule.

Different views have been held as to whether the prices used should be the current prices at the exact date of the valuation, the prices current prior to the date of valuation for the time necessary for the construction of such works, or the average or "normal" prices for a term of years before the valuation, the last method tending to equalize the fluctuations in prices and thereby produce a more stable valuation.

Valuation for rate purposes is generally made to serve for a term of years, and it seems clear that the prices should not be fixed with reference to occasional fluctuations in the prices of labor and materials but rather at the general or prevailing price for labor and materials, which, as a rule, can be deduced by taking the average or prevailing price for a term of years preceding the date of the valuation. From five to ten years has been adopted as a satisfactory period in many cases, but, where charts are available showing the fluctuations of prices of labor and the principal materials entering into a plant, it is well to select the length of the period with a view to obtaining a fair prevailing price.

In some cases, where there is an evident trend toward higher or lower prices, as for instance, the tendency of lumber to increase in price by reason of the diminished supply, and the tendency in the past for cement and aluminum to decrease in price, because of improvements in the methods of and facilities for manufacturing, the prices adopted should be more nearly the price at the time of the inquiry rather than an average of the prices for a period of years.

A Flexible Rail Joint for Steam Shovel Track.

(Staff article.)

Steam shovel track must have considerable flexibility at the joints to meet irregularities in grade and alignment. Ordinary strap joints give this flexibility at the sacrifice of strength and security. The joint illustrated by the accompanying drawings is an improvement over the strap joint; it was devised by Mr. P. J. Thull, a steam shovel operator at Panama, and since about the middle of 1913 has been installed on the track of ten steam shovels—a total of some 220 joints. An official report on the service of these joints to Dec. 30, 1913, is as follows:

> These joints have been in use for several months and are still giving satisfaction.
>
> They are much better than the ordinary strap connection under all conditions, but are especially so in cases where shovels are working on a soft bottom or where there is considerable water in the shovel pit.
>
> The sections when connected with these joints have almost a continuous rail. The old strap connections make such a loose joint that the wheels of the tracks will often climb the rail when moving up, unless great care is taken. This is the case where the sections are down in the mud or covered with water.
>
> These joints are also valuable where shovels are working in slides, where it is often necessary to back the shovel out of the pit very suddenly, without sufficient time available to line the sections up in proper shape."

The construction and operation of the Thull joint are indicated by the drawings. It consists of a male casting *A* and a female casting *B*. These castings are bolted to the rail ends and are hinged by a bolt *C* as shown by the drawings. The hinge *C* provides for joint rotation in the vertical plane; lateral flexibility is provided by the beveling of the socket and other parts by which the two castings engage. A track spike dropped into each of the notches *D* and *E* prevents lateral movement while allowing vertical rotation. The other structural details are plain from the drawings. This joint has been patented.

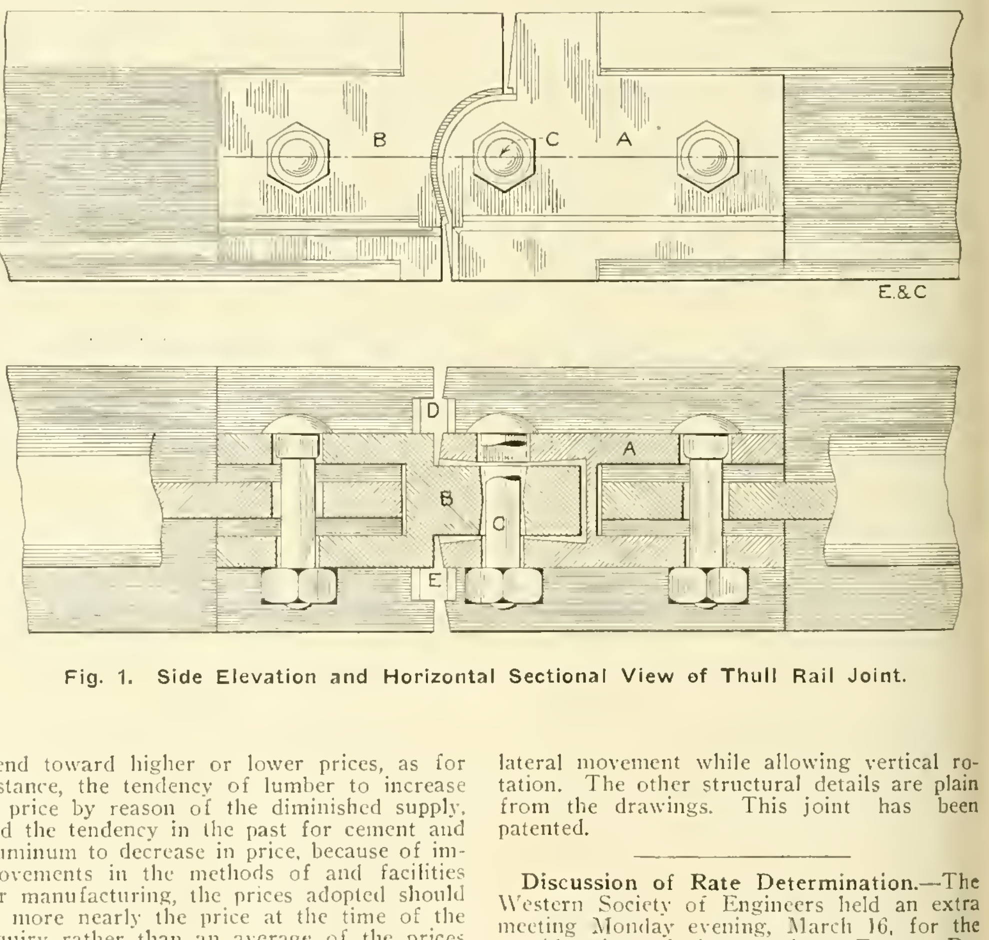

Fig. 1. Side Elevation and Horizontal Sectional View of Thull Rail Joint.

Discussion of Rate Determination.—The Western Society of Engineers held an extra meeting Monday evening, March 16, for the consideration of the question, "Factors Determining a Reasonable Charge for Public Utility Service," which subject was discussed at the annual dinner of the society recently by Mr. M. E. Cooley, dean of the engineering school, University of Michigan. Three short talks were given on various phases of public regulation. Mr. Andrew Cooke discussed "Public Regulation from the Investor's Standpoint." Mr. Samuel O. Dunn, editor of the Railway Age Gazette discussed "Public Utility Valuation from the Railway Point of View," and Mr. Harold Almert discussed "Rate Determination of Public Utilities from the Standpoint of the Student of Public Regulations."

Scientific means of accident prevention will be illustrated at the Panama-Pacific International Exposition by an extensive display in the Palace of Machinery of every device ever invited to protect workingmen. Negotiations have been entered into by the Department of Machinery of the exposition with William H. Doolittle, Safety Inspector of the National Trades Association, and the exhibit will be prepared under his direction.

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*Wilcox v. Consolidated Gas Co., 212 U. S., 19, January 4, 1909.

Engineering and Contracting

Devoted to the Economics of Civil Engineering Design
and to Methods and Cost of Construction

Volume XLI. CHICAGO, ILL., MARCH 25, 1914. Number 12.

Credit for the Dam at Gatun, Panama Canal.

Dead Homer was claimed by seven cities. Not as many worthy contestants appear for the honor of fathering the Gatun Dam, but among those who claim to have begotten the idea and those who helped to nurture its growth this great work can choose a proud ancestry.

In our Panama Canal article of Jan. 7, 1914, we stated briefly the names of those who had suggested the Gatun site for damming the Chagres. Editorially we credited Mr. John F. Stevens with much of the work performed in securing the adoption of this site. In our issue of Jan. 11, 1914, Mr. Isham Randolph, one of the International Board of Consulting Engineers appointed to report upon the type of canal to be built at Panama, questioned the justice of our editorial claim of credit for Mr. Stevens. His recollection was that in the deliberations of the board the first and most urgent advocate of the Gatun site was Mr. Frederic P. Stearns, who himself acknowledged obligation to Mr. C. D. Ward. The same statement of the proceedings within the board has come to us from another member whose name we are not at liberty to publish. This evidence, as we have before acknowledged, unquestionably settles the assignment of credit for securing the interest of the minority members of the Board of Consulting Engineers who finally reported in favor of a lock canal with a dam at Gatun.

The part played by Mr. C. D. Ward in conceiving and nurturing the plan for a dam at Gatun is explained by letter by Mr. Ward himself in another part of this issue. Only one statement of this letter seems to call for any further explanation and then only because in the article of Jan. 7, 1914, we credited Mr. Ashbel Welch with an original suggestion of a Gatun Dam in 1880 when the fact seems to be that Mr. Welch at this time merely spoke in support of the plan as previously suggested by Mr. Ward. Excepting then Mr. Godin de Lepinay, whose paper before the International Scientific Congress at Paris in 1879 proposed a lock canal with a dam at Gatun, Mr. Ward seems to have been the first of record to suggest damming the Chagres River at Gatun instead of at some point much farther inland. Conditions were not propitious in 1879-80 for much consideration to be given to Lepinay or to Ward or, in fact, to anyone who differed from Lesseps. By 1901, however, Lesseps was dead and his sea level canal was a by-word. Then Mr. Ward's contention for a lock canal with a dam at Gatun, plans for which he elaborated in a formal paper before American Society of Civil Engineers, was one likely to impress engineers, and it did, as we have quoted evidence.

But the International Bureau of Consulting Engineers' minority report advocating a lock canal with a dam at Gatun did not appear until 1906. Meanwhile three engineers of national reputation had been in charge of work at Panama. Major Cassius F. Gillette was the first and he was stationed at Panama only during the interval between the departure of the French authorities and the taking over of the work by the second Isthmian Canal Commission. Mr. John F. Wallace was the engineer of this commission during its one year of life. Then followed the third commission, with Mr. John F. Stevens as engineer. Mr Wallace was rather an advocate of a sea level than of a lock canal and he, like the engineers of the first commission (1899-1901) looked upon Bohio or Gamboa as the only suitable sites for a dam to control the Chagres. Major Gillette and Mr. Stevens both, however recorded their preference for a dam at Gatun, and we give in another column Major Gillette's own account of his conception of a Panama Canal plan.

Mr. Stevens' own statement of his advocacy of the Gatun site has been found by us only in his testimony before the Senate committee having under consideration, after the report of the Board of Consulting Engineers, the selection of the type of canal to be built at Panama. On June 23, 1906, Mr. Stevens stated before the Senate committee:

> The first time I went over the canal route I could see the force of the argument very readily about taking care of the Chagres River. I had understood from various writings that Bohio was the lowest point in the valley where the dam could be built. The first thing that occurred to me was: Is it a fact or is it an assumption? And on my way down I noted the narrow place in the valley at Gatun. I immediately asked some of my assistants why Bohio had been chosen for a dam-site instead of Gatun. They said the foundations at Bohio were better than at Gatun. I found, however, that there were no borings at Gatun, and that they did not know anything about it and I commenced to get ready to find out. About that time the Consulting Board cabled down instructions to go on with borings at Gatun—just what I was preparing to do—with the result that we have explained, that in the opinion of everyone, I think, Gatun has proved to be the better site.

With the adoption by Congress of the lock canal plan, the designing of a dam at Gatun was undertaken by the engineers of the fourth and the present Isthmian Canal Commission. The dam as then designed is the dam that now stands completed. One naturally thinks, therefore, of the minority report of the Board of Consulting Engineers as having definitely settled the practicability of the Gatun site. But did it? It was not until Dec. 1, 1907, that the foundation studies were begun which proved the suitability of the Gatun site for a dam and perhaps it was not until the special Board of Engineers was appointed to investigate the proofs and to confirm them that the Gatun Dam became a finally determined part of the Panama Canal plans. We are aware that these last considerations are straining the point, but we introduce them to show that it is not so simple a thing as might be supposed to allot specifically to any one person the credit for this one unique engineering feature of the American Panama Canal—the Gatun Dam and Lake Gatun. Still further when it comes to apportioning the credit due who is entitled to the greatest share? Is it the one who originally conceived the plan, or the one who advocated the plan until recognition was accorded, or the one who provided material proof that the plan was structurally practicable, or the one who designed and built the structures required?

The Ornamentation of Bridges Built As a Part of Track Elevation Work.

In a few of our cities we now have some good examples of what can be accomplished toward beautifying bridges built in connection with track elevation work. In many cities, however, little attempt has been made to improve the appearance of the bridges, and some unsightly structures are the result. The additional cost of bridges which will at least compare favorably in aesthetic design with commercial buildings is not so great as to warrant the construction of bridges such as are to be found in some of our cities. The municipalities can well afford to pay a part of the cost of improved structures, although in most cases the railroads bear the entire cost. The increasing use of concrete, a material which lends itself well to ornamentation provided sufficient care is used in its construction, has done much toward improving the appearance of such bridges. If poorly constructed, however, the good effects produced by the use of concrete are soon lost, and the structures soon present an unsightly appearance. In some cases the demands of residents have resulted in more aesthetic structures, but in many cases the railroads must be credited with having taken the initiative.

On account of the difficulty of securing sufficient clearance without excessive track elevation the through plate girder type of bridge has been commonly used. In their attempts to improve the appearance of these structures engineers have resorted to various treatments of the steelwork. In some cases each girder has been encased in concrete (partly to increase its life), using a paneled effect; in other cases only the outside lines of girders have been encased. For some bridges a concrete fascia and parapet has been attached to each outside girder. In most cases this fascia has had the form of a rectangular beam, while in other bridges its bottom has been curved to give the structure the appearance of an arch, the abutments having been so constructed as to carry out the deception. Various finishes have been used, such as a picked surface, a cement-gun finish, a bush-hammered surface and a sand-blast finish.

In some of the bridges built by the Pennsylvania Railroad Co. for track elevation work in Philadelphia, scroll work and figured designs have first been cast in concrete and then inserted in panels left in the abutments. In one plate girder bridge which is encased in concrete and which is adjacent to a substation colored tiles have been inserted in the concrete surface to form various designs. In general, we believe that this form of ornamentation is out of keeping with concrete bridges, but, due to its proximity to the station, the effect in this case is pleasing. In plate girder bridges encased in concrete great care must be taken to prevent cracks in the concrete casing, as the formation of rust results in streaks on the concrete surface which greatly detract from its good appearance. The design of effective expansion joints between the ends of the girders and the abutments is absolutely necessary if cracks and rust streaks are to be prevented at the ends of girders.

In this issue there appears an article describing the bridges used for the elimination of grade crossings on the Nickel Plate, in Cleveland. Here the problem of ornamentation has been given careful consideration, with the result that some pleasing bridges have been designed and constructed. The views which accompany this article give an idea of the different treatments used for bridges in various localities, the surroundings having much to do with the type of structure adopted. The following types have been used: Three-hinged steel arches with concrete abutments; concrete arches; plate girders with concrete fascias and with the supporting columns encased in concrete; and a composite bridge consisting of a concrete arch and plate girders with a concrete fascia and a false soffit. In some cases the concrete fascia girder is in the form of a rectangular beam with an ornamental iron railing, and in other cases its bottom is curved to give the structure the appearance of an arch and is surmounted by a concrete parapet wall. The ornamental iron railings, however, do not give a good appearance as the lines are too fine.

In the East Boulevard Bridge, which carries two tracks of the Nickel Plate and two of the Short Line, there is a difference of elevation between the two lines of 3.72 ft. There was sufficient clearance on the Short Line side for an arch, but the clearance on the Nickel Plate side was insufficient. The problem was solved and an aesthetic bridge has been obtained by using a reinforced concrete arch for the portion carrying the Short Line tracks and a plate girder bridge for the other half of the structure, the plate girders being masked with a concrete fascia and a false soffit. The structure is faced with a matrix of red granite set in mortar colored with iron oxide, and part of the surface is bush-hammered.

Even though the principal function of a bridge is to carry the loads to which it may be subjected, nevertheless its appearance should be given careful study, especially in view of the fact that engineers are now designing bridges in which "life" is an important factor. In connection with track elevation work, where the spans are usually quite small, bridges which are pleasing in appearance can be constructed at a relatively small additional expense.

Influence of the Engineering Society in Civic Affairs of an Engineering Nature.

Of late certain activities have come to be recognized as among the legitimate functions of the engineering society, such, for example, as the endeavor to secure publicity through newspapers for such portions of the proceedings of the society as are known to be of potential interest to the public. It is our purpose here to discuss briefly another proper function of the engineering society which has seldom been performed namely, the endorsement or condemnation by the voting majority of a society of well-defined projects before a city and of interest to all, or a considerable number, of its citizens. It would appear that as a rule such action can be taken by a society to advantage only with reference to projects before the city in which the society headquarters are located.

An editorial bearing in part upon the subject here under consideration appears in the current issue of the monthly journal of the Engineers' Club of Baltimore. We quote from it as follows:

> Our club, instead of being an active body, unquestionably representing the best engineering talent in the state, and, on many subjects, in the country, has remained passive. This is one of the conditions that should at once be corrected. . . .
>
> When you stop to consider the great number of questions of deep interest to the engineer and citizen that the city is trying to solve, it would seem but natural that this club would let itself be heard and voice some opinion. Surely we are as well qualified to pass on engineering problems, the question of taxation, the fixing of freight rates, the Calvert Street Terminal, the extending of the city's limits, the problem of lighting, the co-ordination of the City Department, safety devices for railroads, etc., as are the Merchants' and Manufacturers' Association, East Baltimore Business Men's Association, Chamber of Commerce, etc., and yet we are never heard from and very seldom heard of, unless it is some little notice to the effect that Mr. Blank read a paper on "Forestry," etc. We cannot expect to grow, either in size or importance, unless we show our hand and let our voice be heard.
>
> We should use our gifts as a power for good in the city and establish for our club the highest respect of the people of Baltimore and be a leading spirit in all public measures. If, when questions are being considered, affecting the welfare of our city, the president would select committees of from five to seven members to report on such questions, and the club at the regular or special meeting either adopt or disapprove the findings of such committees, we would soon be recognized as one of the most important organizations of the city and come into the position to which we are rightfully entitled.

What is said in the foregoing quotation relative to the Engineers' Club of Baltimore can be stated with equal truth of other engineering societies and clubs in all the larger cities of the country. In saying this we do not endorse the expressed views in full, for we consider that perhaps too many matters are enumerated as of sufficient interest to the club as a whole to warrant their consideration by the full membership. The general principle laid down—namely, that the local engineers' club should seek to exert influence as an organization in certain selected engineering projects before the city—is sound and is worthy of a thorough trial by such organizations throughout the country.

In speaking of selected projects we have in mind the necessity of limiting the censorship of the society to projects of major importance and of very wide public interest. It seems to us that when a complete and formal report is made upon such a project to the city's governing body that the society would do well to discuss the report in open meeting, in the presence of the authors of the report if that be desired, and then take action either in endorsement or condemnation of the project in its broad engineering and economic aspects. Perhaps the directors or a special committee of the society should pass upon the importance of various engineering projects before the city, with a view of submitting those of greatest importance to the society for the character of consideration here under discussion.

We believe that the composite judgment of the society, as indicated by a vote, would not only give a true estimate of the worth of a project, but would be so accepted by the most influential people of the community.

Adam Smith vs. Henry George On Wages.

The father of political economy, Adam Smith, says in "The Wealth of Nations":

> England is certainly, in the present times [1776], a much richer country than any part of North America. The wages of labor, however, are much higher in North America than in any part of England. In the province of New York, common laborers can earn three shillings sixpence currency, equal to two shillings sterling [or 50 cts.] a day; house carpenters and bricklayers, four shillings and sixpence [or $1.10]. * * * The price of provisions is everywhere in North America much lower than in England. * * * Eighteen pence [32 cts.] a day may be reckoned the common price of labor in London and its neighborhood. A few miles distance it falls to fourteen and fifteen pence. Ten pence [20 cts.] may be reckoned its price in Edinburgh and its neighborhood. At a few miles distance it falls to eight pence [16 cts.], the usual price of common labor through the greater part of the low country of Scotland.

From this it appears that common laborers in America in 1776 received two to three times as high wages as in England and Scotland. About the same ratio still exists. We noted in our issue of March 4 that bricklayers in New York City are asking an advance to $6 a day, the present wage being $5.60. Incidentally let it be noted that this is five times the daily wage of bricklayers "in the province of New York" in 1776. Frequently it is said a dollar went so much farther a century or more ago that no fair comparison can be made between wages then and now. There is, however, little truth in this belief. Adam Smith gives tables of the price of wheat in Great Britain from the year 1200 down to 1764, showing a remarkably constant price of about $1.25 per bushel (or higher than in 1914), the general tendency being toward a lower price. He says:

> During the course of the last century, taking one year with another, grain was dearer in both parts of the United Kingdom than during that of the present. * * * In the last century the most usual day-wages of common labor through the greater part of Scotland were six pence [12 cts.] in summer and five pence [10 cts.] in winter.

He comments on the much improved status of the laborer in 1776 as compared with previous times.

In 1877, almost exactly a century after Adam Smith's great work was published, another remarkable book on political economy was written We refer to Henry George's "Progress and Poverty." Although Henry George was unquestionably wrong in his contention that the average workman had not benefited by the progress of invention, he was right in his criticism of Adam Smith's "wage fund theory." Ignoring the testimony of wage and price statistics, Henry George fancied that poverty was steadily increasing amid advancing wealth. To quote:

> In England there were [1877] over a million paupers. In the United States almshouses are increasing and wages are decreasing.

Wages in New York have increased from 50 cts. for laborers in 1776 to $2 in 1914, and from $1.10 for bricklayers in 1776 to $5.60 in 1914. The working day has two-thirds as many hours now as it had a century ago.

Adam Smith repeatedly recognized the influence of invention as an important factor in raising wages, but he put much greater stress on the "division of labor."

Henry George, curiously enough, mentioned neither of these two factors, but explicity limited the means of securing higher wages to the following methods:

> I. From greater economy in government.
>
> II. From the better education of the working classes and improved habits of industry and thrift.
>
> III. From combinations of workmen for the advance of wages.
>
> IV. From the co-operation of labor and capital.
>
> V. From governmental direction and interference.
>
> VI. From a more general distribution of land.
>
> Under these six heads I think we may in essential form review all hopes and propositions for the relief of social distress short of the simple but far-reaching measure which I shall propose.

George reviews each of the six methods and dismisses them in turn as being, at best, palliatives. "We must make land common property," he reasons, if wages are to rise.

Land has remained private property, yet wages have risen enormously in the 37 years that have elapsed since Henry George wrote. Moreover there must be some cause for the rise in wages other than any of the six that George mentions. The last century, and particularly the last fifty years, differs mainly from all previous periods in the development of industries wherein engineering plays a prominent part. Transportation costs have decreased greatly, and nowhere to such a degree as in America. The cost of power has decreased quite as rapidly. Only the other day quite a tempest of discussion was caused among electrical engineers by the reading of a paper in which a cost of one-quarter cent per kilowatt hour was given as being attainable with steam power and modern turbo-generators. While this low cost is undoubtedly not within reach as yet, except under abnormal conditions and probably not then, it may possibly be attained within, say, twenty years if progress shall continue at its present pace.

The very growth of power consumption—which has been about as the square of the growth of population—is an important factor in lowering the cost of power. Not only do large turbo-generators yield power at lower cost than small units, but higher load factors further reduce the fixed charges per unit of power. Inventions multiply on every hand, and they are able to multiply because there is little resistance, on the part of American workmen, to the introduction of labor saving devices. But inventions are not the sole cause of rising wages. American managers have been foremost in recognizing the greatest single economic principle in fixing wages, namely that if men are paid in proportion to their efficiency they will steadily increase their output. In this connection it is interesting to note the century old remarks of Adam Smith bearing on this point:

> The wages of labor are the encouragement of industry, which, like every other human quality, improves in proportion to the encouragement it receives. * * * Where wages are high, accordingly, we shall always find the workmen more active, diligent and expeditious, than where they are low. * * * Workmen, when they are liberally paid by the piece, are very apt to overwork themselves and to ruin their health and constitution in a few years.

This last statement about piece-work applies more particularly to men who have been very meagerly paid on a wage, and whose work involves heavy muscular effort. Neither of these conditions exists to any great extent in America. Hence piece-work and bonus payments are becoming progressively more popular in this country.

It is interesting to conjecture what Henry George would have thought of his prediction of progressive diminution in wages had he lived to read the recent announcement that an American automobile factory pays even its office boys $5 a day. This, while an unprecedented and economically unfair wage for such service, at least points the goal toward which common labor wages in America are trending.

The final cause of the higher standard of wages is engineering invention which has made such a standard possible. Higher wages prevail and portend not because of labor unions, nor because of "single taxes," nor because of public ownership of capital, but mainly because of engineering. We use the term engineering in the broad sense that the modern development of engineering science justifies.

General

Credit for the Dam at Gatun in Planning the Panama Canal.

To the Editors: I thank you for having sent me a copy of your valuable paper of Jan. 14th containing, what was to me, a very interesting article on "Credit for conceiving the plan of the dam at Gatun for a Panama Canal."

It might be interesting in this connection to look over an article on the "Gatun Dam" in the sixth edition (1911) of Edward Wegman's Book on Dams. On page 453 he says, "The fact that the construction of the Gatun Dam had, at last, been decided upon—27 years after it had been proposed by Lepinay, and so soon after Mr. Ward's forceful argument in its favor—would indicate that considerable credit is due to Mr. Ward for the final adoption of this very important feature of the Panama Canal."

"Engineering News" of Mar. 8, 1906, page 267, also discusses the same subject.

The Isthmian Canal Commission of 1899-1901, appointed by the government, after studying the canal question for two years and eight months, recommended in their report, the Panama Route for the canal with a dam at Bohio to hold the water of the Chagres River at a mean summit elevation of 85 ft. above tide. They also in their report, without reservation or qualification, said that, "No location suitable for a dam exists on the Chagres River below Bohio.

In 1901 my paper was read before The American Society of Civil Engineers of New York, entitled "The Gatun Dam," which was virtually a criticism of the 1901 report as to the proposed location of the dam at Bohio, and in it I strongly advocated the advantage of the Gatun location instead of Bohio, with the same summit elevation and urged that examinations of that site should be made before deciding on the final plans.

The effect of that short paper was to lead to a very thorough examination of the site by borings, etc., by the Isthmian Canal Commission to determine its suitability for the site of the great dam. The result of the examination was so favorable that the plan received consideration and recommendation from the minority (5 out of 13) of the Board of Consulting Engineers of 1905. The acceptance of the lock plan, based upon the dam being built at Gatun, by the Secretary of War and the President, and its final adoption by Congress followed duly and the dam has been built on that location making a lake of 164 square miles instead of a much smaller one of 85 sq. miles, as it would have been if the dam had been built at Bohio, nine miles farther up stream.

A dam at Gatun had been suggested to the Lesseps' Paris Congress in 1879 by M. Godin Lepinay, and also to the American Society of Civil Engineers by Mr. Ashbel Welch [illegible] in which Mr. Ward concurred, though neither had then heard of Lepinay's plan, but the plan had never been followed up and was soon dropped, either as impracticable or supplanted by the project for the dam at Bohio. The revival of this dead and forgotten plan, in the opinion of at least four members of the Consulting Board of Engineers of 1905, was due to my paper of 1904. As has been said by one well fitted to judge, "While it is impossible to say that the Gatun location would not have been adopted had Mr. Ward's paper not been written, the facts that the first suggestion to this end was dropped and remained unheeded except by him for more than 20 years and that its revival was due to him exclusively make it most probable that but for his persistant efforts the excellent plan of canal now adopted would not have developed and either an inferior lock plan or a far more costly sea-level plan adopted."

The difference in cost between a sea-level canal and the one built would have been about two hundred million dollars.

The paper, "The Gatun Dam," of 1904, was I believe the first to advocate the location of the dams and locks at the south end of the canal, so near to the Pacific Ocean at La Boca, later called Balboa, thus forming a lake there five miles long. This location was also adopted by Congress but was later changed for military reasons and anticipated difficulty in construction, to Miraflores, where the 1899 report had located them. The Special Board of Engineers of 1909 say in their report, among other things, that they were unanimously of the opinion that the dams could have been constructed at La Boca and that the change of location would involve an additional expense of not less than $10,000,000. (See "Canal Record" of Mar. 3, 1909).

Gatun Lake, with its area of 164 square miles, is believed to be the largest artificial lake in the world, if this is not the case, should be glad to know of the larger one.

Yours respectfully,
C. D. Ward.

702 St. Nicholas Ave., New York, Feb. 5.

To the Editors: I note your very excellent article on the Panama Canal in a recent number.

May I ask that you devote a little time to investigating who is responsible for the adoption of the key of the situation, the Gatun Dam? If you will read with some care my article in the "Engineering News" of about July 27, 1905, the first article in the number, I think you will note that the plan there recommended was essentially the one adopted and I believe was the determining factor in the situation.

The only essential changes made in the minority report and the plan finally adopted were:

(1) To reduce the crest from 100 ft. to 85 ft. They were needlessly nervous over going much beyond the height they adopted and it has since been claimed that 85 ft. carried out the principle for which I contended, that there should be slack water a mile up every tributary to the narrow part of Gatun Lake to help take care of the sediment. This question of sediment is amplified in my paper, the first time I think on record, the subject having been previously dismissed by the statement that the Chagres was a clear water stream, yet Colonel Goethals built one or two locks out of the sand and gravel on one of the bars of that clear water stream, renewed in great quantities after every flood. The sediment would have been a tremendous obstacle to a sea-level canal. As to the relative merits of 85 and 100 ft. elevation for the dam, I think the last 15 ft. excavation in Culebra Cut developed many millions of yards of slide, which could have been avoided. I also recommended caisson gates instead of the mitering lock gates, to which latter proposition can be atributed the fear of greater lift.

(2) I recommended a single flight of three locks at Gatun with space arranged for a second flight on larger dimensions as shipping developed. Ease of enlargement is the important element in the canal. Puting in the double flight of abnormally large locks now I think is something of a mistake vastly increasing the cost, making inconvenient future enlargement and only saving a few hours, which, in a 5,000-mile journey, does not amount to much.

(3) My plan provided for a 2,000-ft. self-regulating concrete spillway with the locks on the rocky "island" in the midst of the alluvial plain. The adopted plan moves the locks ashore, puts a man-regulated spillway on the "island" with a longer dirt dam replacing the dirt dam of my plan and extending it in place of the 2,000-ft. concrete spillway.

These are all minor changes, none of them I think to the benefit of the proposition.

If you will read carefully the various reports, you will find that they adopted essentially my plan, but have for some reason which I have never discovered sidestepped giving me credit for it, although they gave me specific credit for two very important matters, the straight canal from Gatun through the center of Limon Bay instead of the crooked channel that the French had started, and the demonstration of the futility of the high dirt dam with concrete core which the sea level canal people proposed at Gamboa.

Upon my verbal recommendation Secretary Taft threw out bids which the commission had obtained to spend $8,000,000 for the improvement of the crooked canal in the vicinity of Colon.

Your article gives credit to Mr. John F. Stevens for discovering the value of the Gatun site, but if you will get the dates on the subject you will also see this suggestion was long after my paper was published. In that paper I gave credit to Mr. C. D. Ward for having expressed an early thought on the subject and I believe the American Society of Civil Engineers, for some curious reason, gave full credit in the matter to one of its former presidents, Mr. Ashbel Welch, who gave some approval as I recall it to Mr. Ward's idea, but I believe I am correct in stating that neither of them ever saw the site or made any plans for a dam or did anything really to get that site adopted. So far as I know, if anybody is accepted as "Godfather" to that interesting subject, I think it should be your humble servant.

Very truly yours,
Cassius E. Gillette.

[illegible] S. Twentieth St., Philadelphia, Pa., Jan. 31, 1914.

Waterproofing of Masonry and Bridge Floors.

Waterproofing materials and methods have been subjects of careful investigation for several years by a committee of the American Railway Engineering Association. This committee has from year to year presented compilations of data, but its final report on waterproofing masonry and bridge floors has only recently been completed and presented to the association. We believe that the final report is one of the best summaries of practical waterproofing materials and methods that is available to engineers and we therefore publish it slightly condensed as follows:

Structures should be waterproof when it is necessary: (1) To prevent dampness in walls above grade, and in walls and floors below grade. ((2) To prevent flooding of basements and pits which are at all times or occasionally below the ground water level. (3) To prevent percolation or leakage of water through the masonry and the formation of unsightly deposits on exposed surfaces. (4) To prevent the dripping of water through a bridge floor over a street, and in the cases of solid floors of steel or reinforced concrete bridges, to protect the steel from corrosion. (5) To prevent the entrance of water into tunnels, either above or below ground water level, or subaqueous tunnels. (6) To prevent leakage from reservoirs. (7) To prevent the penetration of water into the masonry.

The following outline includes the ordinary methods of waterproofing:

(I) Coatings: (1) Linseed oil paints and varnishes. (2) Bituminous: Asphalt coal tar. (3) Liquid hydrocarbons. (4) Miscellaneous compounds. (5) Cement mortar

(II) Membranes: Felts and burlaps in combination with various cementing compounds.

(III) Integrals: (1) Inert fillers. (2) Active fillers.

(IV) Watertight concrete construction.

Walls above grade are waterproofed by coating with paints, varnishes, or waterproofing washes, or by plastering with cement mortar. The coating or plaster may be applied either on the inside or outside of the wall.

The walls of basements and pits are waterproofed, either by the application of coatings, membranes, integral or watertight concrete construction. Membranes are usually protected with concrete, brick or bituminous binder.

Where basement or pit walls and floors are below the ground water level, they must be so designed as to resist the existing hydrostatic head in order to prevent cracks and leakage. Such walls may be waterproofed by the integral method or by watertight concrete construction. When exterior waterproofing is employed, the membrane method is generally used properly protected.

Stone, brick or concrete arches, retaining walls, abutments, subway walls and culverts are waterproofed by any of the methods mentioned in the preceding paragraph. For important structures, the membrane method is most generally used.

When surface coatings, integral waterproofing or watertight concrete construction is used, particular attention must be paid to reinforce the work against cracks due to expansion, contraction or settlement. The expansion joints must be waterproofed by sheet copper or lead built into the adjoining sections.

The solid floors of steel and reinforced concrete bridges probably present the most difficult problems of waterproofing. In steel troughs or I-beam floors a concrete filling may be used to bring the deck up level with, or above the top of the steel in the floor. The floors of this class of structure are usually waterproofed by the membrane method.

Tunnels in which the ground water level is below the invert may be waterproofed by any of the aforementioned methods.

Subaqueous tunnels present a different and distinct problem of waterproofing; usually reinforced concrete, or plain concrete, with iron or steel lining is used. The structures are designed to resist the hydrostatic head.

The walls and floors of reservoirs may be waterproofed by any of the four methods before mentioned.

COATINGS.

Linseed Oil Paints and Varnishes.—Linseed oil paints and all coatings containing linseed oil are reactive to atmospheric conditions and to alkaline water. Applied as a dampproofing to the surface of a concrete wall which may be permeable to moisture, the paint is likely to be of short life unless the surface is specially prepared. To secure the best results, the wall must be dry and clean before application. The paint is applied with a brush in the ordinary manner. The coating power of paint is approximately 200 sq. ft. of wall per gallon of paint, but varies with the thickness of the paint and the nature of the surface. The prices of the paints sold for dampproofing masonry and concrete surfaces vary from about $1 to $3 per gallon for the material.

Bituminous Coatings.—This class includes asphalt, petroleum residiuum, coal tar and coal tar pitch. As used for waterproofing purposes, they are solid at ordinary temperatures and are, therefore, often applied while hot. As they are soluble in benzine and coal tar naphtha, they are frequently mixed with these solvents and applied in a liquid form. Two coats cost about 1 ct. for material and ¼ ct. for labor per square foot.

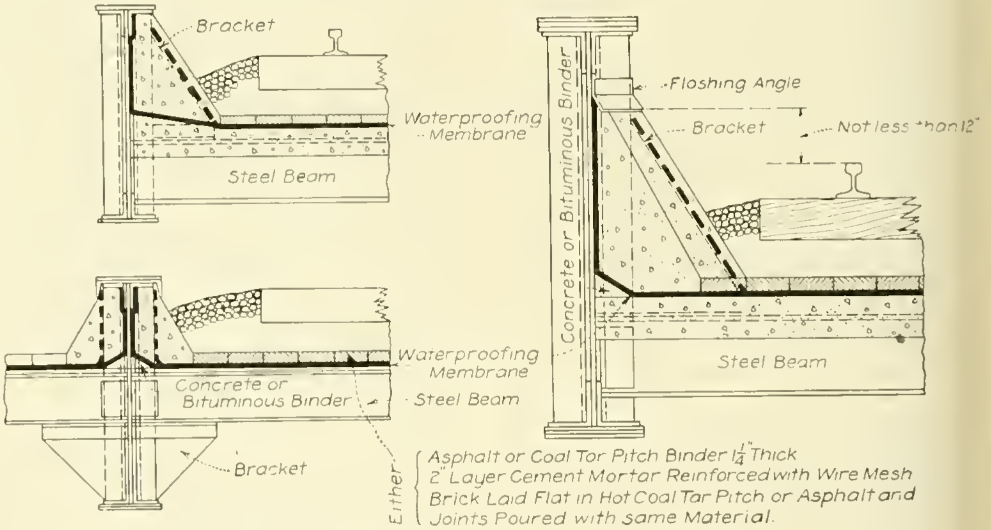

Bracket
Waterproofing Membrane
Steel Beam
Waterproofing Membrane
Concrete or Bituminous Binder
Steel Beam
Bracket
Concrete or Bituminous Binder
Floshing Angle
Bracket
Not less than 12"
Steel Beam
Either
Asphalt or Coal Tor Pitch Binder 1¼" Thick
2" Layer Cement Mortar Reinforced with Wire Mesh
Brick Laid Flat in Hot Coal Tar Pitch or Asphalt and Joints Poured with same Material.

Fig. 1. Methods of Waterproofing Plate Girder Bridge Floors.

Asphalt.—Waterproofing by the application of liquefied asphalt, as a paint applied with a brush or mop, has been used on practically all kinds of engineering structures as a surface coating.

Bituminous coatings applied cold by dissolving in naphtha, instead of hot, do not set instantly, therefore are much easier to apply. The work can be done by an ordinary laborer, care rather than skill being required in its handling. All walls that are to be waterproofed must first be allowed to dry.

If the waterproofing is made by dissolving the bitumens in a volatile solvent with a dryer so that it may be applied cold like a paint, it is difficult, if not impossible, to prepare a paint that will dry to the right consistency and then stop. The usual result is that the drying and hardening continues until it reaches a point where its waterproofing qualities are destroyed.

Hot asphalt will not adhere to cold, damp concrete. Several different methods of heating the surface of the concrete have been used. Gasoline has been poured over the surface and burned; hot sand has been spread over the surface and swept back as the waterproofing proceeds. It is claimed, however, that heating the surface draws up moisture and prevents the asphalt from adhering. It is necessary that the concrete be thoroughly dry before the asphalt mixture is laid upon it, as the steam caused by placing the hot material upon a damp foundation will prevent adhesion. Good results have been obtained by first painting the surface to be treated with a priming coat of asphalt cut with naphtha or benzine and then applying the hot asphalt over this coat.

In applying hot asphalt directly to steel, difficulty is found in getting the asphalt to adhere to the steel, and no dependence can be placed upon adhesion to vertical surfaces.

The asphalt should be heated in a suitable kettle to a temperature not exceeding that allowed in the specifications for any particular structure depending upon the material used. If this temperature is exceeded, it may result in pitching the asphalt. Before the pitching point is reached, the vapor from the kettle is of a bluish tinge, which changes to a yellowish tinge after the danger point is exceeded. The asphalt has been cooked sufficiently when a piece of wood can be put in and withdrawn without the asphalt clinging to it. Care should always be taken not to prolong the heat to such an extent as to pitch the asphalt. Should it become necessary to hold the heated asphalt for any length of time, the fire should be drawn or banked and a quantity of fresh asphalt should be introduced into the kettle to reduce the temperature. Excessive heat converts the petroline or cementitious constituents of the asphalt into asphaltene, which is devoid of cementing properties and by so much reduces the cementing quality—the vital element—of the asphalt. The fire should not be allowed to come into direct contact with the melting kettle or tank. Asphalt coatings cost about 65 cts. per gallon for material and 3/10 ct. for labor per square foot, a gallon covering about 100 sq. ft. per coat.

Asphalt Mastic.—Various results have been obtained by the use of asphalt mastic, and it is probable that much is dependent upon the quality of the mastic. The requirements of a sand for asphalt mastic are much the same as those for cement mortar. It is common practice to mix a certain amount of limestone screenings with the sand, with the intention of securing an aggregate with the least percentage of voids. The strength and compactness of the mastic will depend considerably upon the percentage of voids, and the proportion of asphalt used in the mastic should be sufficient to fill the voids and completely coat each particle of sand and screenings. Too much asphalt will produce a mastic that is soft and easily indented, does not offer a good protection against the ballast on a bridge floor and flows more readily than a well-proportioned mixture.

The asphalt and sand are separately heated to from 325 to 350 degrees. The proper proportions are measured out simultaneously, poured into a mixing vessel and thoroughly mixed. The operation of mixing the asphalt mastic requires care in heating the ingredients to secure uniform temperature, not to overheat the asphalt, to proportion the mixture accurately, and to mix the materials thoroughly. The mixture is dumped in place and spread evenly over the surface with wooden floats, shovels or rakes. After being compressed with tampers, the surface is finished with hot smoothing irons.

Asphalt mastics are usually applied in layers not exceeding ⅝ in. in thickness. Usually two coats are applied, the coats to break joints not less than one foot. The cost of asphaltic mastic 1¼ in. thick is about $30 for material per net ton, a ton covering about 375 sq. ft.; the cost of labor is about 2 to 5 cts. per square foot, depending upon location and conditions.

Coal Tar and Coal Tar Pitch.—Tar produced by the distillation of bituminous coal is used in waterproofing, either applied cold as a paint by dissolving in naphtha or benzine or applied hot. It is also mixed with sand, gravel or screenings to form a mastic. It is generally found to be difficult to obtain coal tar of good quality. Good coal tar compares favorably with asphalt as a waterproofing material. The present price of coal tar pitch used for waterproofing is about $17.50 per net ton.

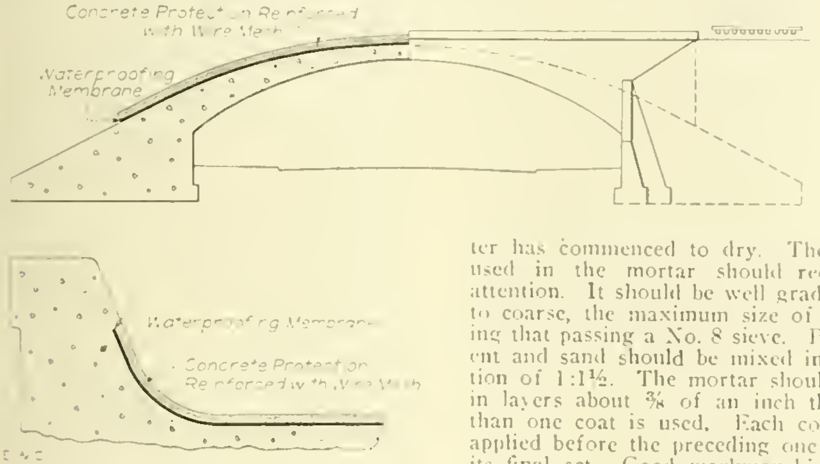

Concrete Protection Reinforced with Wire Mesh
Waterproofing Membrane
Waterproofing Membrane
Concrete Protection Reinforced with Wire Mesh

Fig. 2. Method of Waterproofing Concrete Arch Bridges.

Coal Tar Paint.—Annapolis mixture is a coal tar paint composed of 1 part kerosene oil, [illegible] parts Portland cement and 16 parts refined coal tar. The mixture is put on with a paint brush in the same way as ordinary paint is applied. The compound not only covers the surface, but sinks into and bonds with it, so that two or three coats are sometimes required. It has been found to adhere to moist or even wet concrete. The cost for three coats is about ½ ct. for material and about ½ ct. for labor per square foot.

Liquid Hydrocarbons—Paraffin and Petroleum. Waterproofing by the application of a coating of melted paraffin has been used on masonry in much the same manner as hot asphalt. Paraffin is also applied cold as a paint made by dissolving the paraffin with naphtha. Petroleum oil is sometimes applied to the surface of masonry as waterproofing. The efficiency of these materials depends upon their absorption into the surface of the masonry. Applied to clean, dry surfaces of porous masonry, they are fairly efficient as damp-proofing.

Soap Washes.—Solutions of soap applied as a wash for waterproofing or damp-proofing masonry surfaces are not recommended, as no permanent waterproofing effect can be depended upon.

Soap and Alum Washes.—Waterproofing by alternate washes of soap and alum is one of the oldest methods of treating masonry surfaces, and has given fair results when properly used on surfaces sufficiently dense and impermeable to afford support for the void filling material. Inferior materials and workmanship cannot be atoned for by the use of alum and soap washes. The alum and soap combine and form an insoluble non-absorptive compound in the pores of the masonry surface. The cost of applying two coats each of soap and alum washes is about ½ ct. per square foot of surface.

Cement Mortar.—The method of waterproofing masonry structures by the application of a plaster coat has proved efficient when the plaster has been properly applied. The surface to be waterproofed must be clean to insure bond between plaster and masonry. Old surfaces may be cleaned by chipping off a thin layer from the face or by the use of a sand blast or steam jet. The surface must then be kept wet until it has absorbed water to its full capacity. A wash of neat cement mortar should then be applied with a brush. This wash should be mixed to the consistency of cream and should never be used after it is 45 minutes old. The plaster should be applied over the cement wash before the latter has commenced to dry. The sand to be used in the mortar should receive careful attention. It should be well graded from fine to coarse, the maximum size of particles being that passing a No. 8 sieve. Portland cement and sand should be mixed in the proportion of 1:1½. The mortar should be applied in layers about ⅜ of an inch thick if more than one coat is used. Each coat should be applied before the preceding one has attained its final set. Good workmanship is essential and the use of a wooden float is necessary in order to obtain a dense, impermeable coating. As ordinarily applied, the finished coating is about ¾ of an inch thick. The cost of ¾-in. plaster, applied as above, will be about 6 cts. per square foot.

MEMBRANES.

Membrane waterproofing consists of the formation of a mat or covering of waterproofing material over the surface to be waterproofed, made up of a number of layers of membrane united by a cementing material. Being somewhat elastic and independent of the movement of the surface, this method offers a protection from the seepage of water through expansion or contraction joints and cracks in the masonry which cannot be secured by any other. For this reason it is largely used for waterproofing subways, arches, solid floor bridges, retaining walls, basements, pits, etc. It is also largely used in important structures in connection with some integral form of waterproofing as a precaution against seepage of water through cavities that may occur in the masonry.

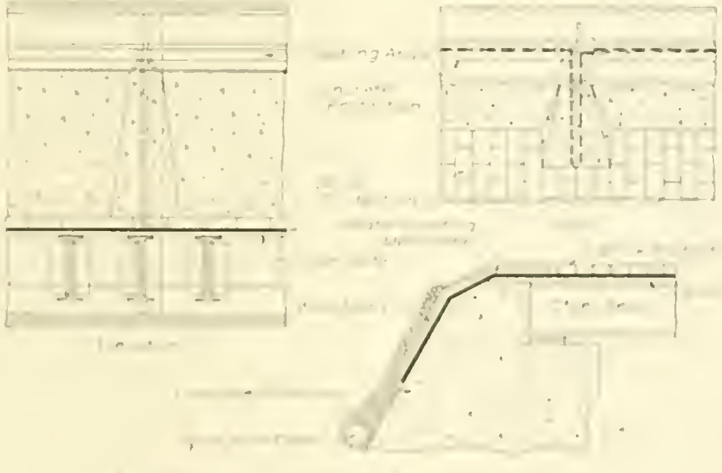


Fig. 3. Method of Waterproofing Steel Beam Bridge Floors.

Although waterproofing by the membrane method has been unsuccessful in many cases and many reports of failures are returned by the railroad companies, the better methods of membrane waterproofing now in use are giving excellent results. The character of the structure is frequently the greatest drawback to the life of the waterproofing. The greater the number of projections and irregularities in the surface to be waterproofed, the more the liability of leaks. Many times the design of the structure is such as to make it impracticable to waterproof in a permanent manner. Sudden slopes or deep drops between the different elevations of the floor often cause the protection to slide, with a consequent tearing of the waterproofing. Often on railroad bridge floors the waterproofing is destroyed by the creeping of its protection under traffic; on arches or sharply inclined surfaces by its movement due to the settlement of the fill. In many cases the labor employed is quite unskilled and the results are obviously poor. Another factor in the success or failure of waterproofing is the state of the weather. In cold weather the heated materials cool too rapidly. In very damp or rainy weather it is impracticable to make a good job of waterproofing, unless some protection from the weather is provided. Other causes of failure are the lack of free working space and interruption by traffic. Any of these causes may lead to failure, even with the best materials.

Materials.—The materials of membrane waterproofing and the combinations that have been used most successfully by the various railroads are as follows:

Wool felt impregnated with either asphalt or coal tar pitch.

Wool felt impregnated with either asphalt or coal tar pitch and skin coated with the same material.

Wool felt impregnated with coal tar pitch and reinforced with a thickness of cotton drilling cemented to the felt with coal tar pitch.

Asbestos felt impregnated with asphalt.

Burlap both plain and impregnated with either coal tar pitch or asphalt.

Mined or lake asphalts.

Petroleum asphalts.

Coal tar pitch.

Two to three layers of felt cemented together, used generally for damp-proofing and for the backs of retaining walls or foundations where no provision for a head of water is necessary.

Four to six layers of felt cemented together, used generally for railroad bridge floors, arches, tunnels, subways and for a protection from a head of water.

To add tensile strength to the waterproofing, the following combinations are commonly used:

One middle layer of reinforced felt or burlap and four layers of felt, all cemented together.

One layer of felt, two layers of burlap and two layers of felt cemented together.

Three layers of burlap and one top layer of felt cemented together.

Combinations of coal tar pitch and asphalt treated felt or asphalt and coal tar treated felt should not be used as the materials will not combine.

In using burlap it is recommended that burlap impregnated with either asphalt or coal tar pitch be used, otherwise, owing to its nature, it is impracticable to prevent the absorption of moisture when the material is exposed to the weather. Moisture promotes rot and also greatly reduces, or, if present in any quantity, prevents the bond of the hot cementing material and its penetration of the pores of the burlap. On the other hand, the treating of burlap promotes the bond and penetration as the treating materials in the burlap are softened on the application of the hot cementing material, and the whole becomes united in one mass.

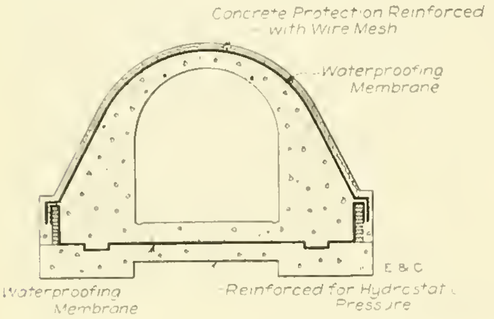

Concrete Protection Reinforced with Wire Mesh
Waterproofing Membrane
E & C
Waterproofing Membrane
Reinforced for Hydrostatic Pressure

Fig. 4. Method of Waterproofing Concrete Lined Subway.

The use of burlap with cementing material, whose temperature on application exceeds 450° F., is not recommended, as the higher temperatures are likely to result in burning and destruction of the burlap.

In many cases it is desirable to bond the waterproofing to the surface. This is not desirable in the vicinity of expansion joints or where there is likely to be a movement of the surface. At such points special provision must be made in the waterproofing to allow for expansion.

Protection.—To protect the membrane from injury it is necessary to provide a covering of some hard material that cannot be penetrated by ballast, tamping picks nor by sharp stones. Of the various methods, the following three have been the most widely used:

(1) Brick laid flat in the hot cementing material with joints poured with the same material, or brick laid in cement mortar. On comparatively flat surfaces, brick is practicable with a bituminous binder, but on steep surfaces or slopes, the tendency to creep in hot weather makes it unsuitable. One great advantage of brick is that it can be laid quickly and easily under traffic. Brick, if used on large areas or on the extrados of an arch or on steep slopes, should be laid in cement mortar to prevent creeping.

(2) A cement mortar coating about 2 ins. thick, reinforced with wire mesh, forms a good protection and can often be used to better advantage where there is a tendency of the protecting materials to creep. This protection is recommended for arches and tunnels

(3) A bituminous binder not less than 1¼ ins. thick, consisting of asphalt or pitch mixed with sand, gravel or fine crushed stone and applied over the waterproofing, has often been successfully used. If this is used, it should be of such consistency in hot weather as to prevent runs and the stones forcing through the protection to the waterproofing. It is not recommended on steep slopes.

Specifications.—The following specifications for five-ply waterproofing is typical of those in use by the various railroads, and applies equally well to combinations of felts and burlaps or felts and reinforced felts:

The surface on which the waterproofing is to be applied shall be dry and free from all sharp projections or irregularities of any character other than those shown on plans.

If it is desired to secure the waterproofing to the surface this surface shall be given one coat of hot cementing material mopped on uniformly, which coating shall be thin enough to penetrate the recesses, and in the case of concrete, to form a bond for the subsequent waterproofing coating. In order to insure the adhering of this coating it is advisable, in cold weather, to first heat the surface with hot sand, which is to be swept off as the cementing material is applied, or a priming coat of the cold cementing material which has been thinned with a suitable solvent may be applied.

On this first coat shall be applied a heavy coating of hot cementing material, into which shall be laid, shingle fashion, two layers of felt lapped one-half the width of the felt and cemented together with cementing material. The surfaces of the two-ply felt thus formed shall be mopped uniformly with hot cementing material and followed with three layers of felt laid shingle fashion in this material and lapped two-thirds of its width. The surface of the five-ply felt thus formed shall be given one heavy coat of cementing material, making a five-ply waterproofing membrane all thoroughly saturated, cemented and bonded together.

In the courses thus built up it is important to have the moppings of cementing material uniform, so that felt shall not touch felt at any point and to insure a surface free from all folds and pockets.

At girder webs or around gusset plates, corners, or over column connections and expansion joints, the waterproofing membrane shall be reinforced with at least two thicknesses of felt.

Over the surface of the membrane shall be placed a protection of either brick, bituminous binder or concrete, plain or reinforced.

Cost.—Cost of membrane waterproofing varies greatly with conditions. A five-ply membrane waterproofing, with asphalt-treated felts cemented with asphalt, will cost from 25 cts. to 45 cts. per square foot, including a bituminous binder or brick protection and labor. A five-ply membrane waterproofing, using four layers of coal tar pitch-treated felt and one layer of felt reinforced with cotton drilling, cemented with coal tar pitch, will cost from 20 cts. to 35 cts. per square foot, including bituminous binder or brick protection and labor. A four-ply membrane waterproofing, using one layer of asbestos felt and three layers of impregnated burlap cemented with asphalt, including 1¼-in. thick asphalt mastic protection and labor, will cost from 20 cts. to 30 cts. per square foot; cost of asphalt about $30 per gross ton; cost of coal tar pitch about $17.50 per gross ton; cost of asphalt treated felts from $1 to $1.25 per 100 sq. ft.; cost of coal tar pitch treated felts about 25 cts. per 100 sq. ft.; cost of reinforced felt from $2 to $2.25 per 100 sq. ft.; cost of asbestos felt about 70 cts. per 100 sq. ft.; cost of brick $8 to $12 per thousand.

INTEGRALS.

The use of some material in small quantities, mixed with the concrete materials in order to make concrete watertight, is generally called the integral method of waterproofing.

Inert Fillers.—The addition of a small amount of fine material to a rich concrete mixture with a well-graded aggregate, decreases the strength of the concrete. The effect upon leaner mixtures is to increase the impermeability of the concrete without decreasing its strength. Fillers used should not only be inert toward the action of the cement, but also to atmospheric conditions and to water. Material containing organic matter should be avoided, owing to its deleterious effect upon the strength of the concrete.

In using inert fillers in mixing concrete only such materials should be used as have been thoroughly analyzed as to their chemical properties and effect upon the concrete both as to strength and chemical action. The amount of inert fillers used must be determined by careful tests.

The waterproofing effect of inert fillers depends upon the void-filling quality of the material used and upon the grade of workmanship insisted upon; the addition of a waterproofing compound to the concrete material coupled with poor workmanship will not assure watertight concrete.

It is an open question whether it is good engineering, especially on important structures, to omit precautions and methods of workmanship, which improve the quality of the resulting concrete in any respect, in order to reduce the cost and produce a somewhat inferior concrete which meets the present needs. There is a possibility that in gaging the amount of money to be spent in making concrete by the strength required, other factors may be lost sight of which may in time prove harmful to a structure which was supposed to be of the most durable construction.

There are numerous examples on record where structures have been built of concrete, in the too often used haphazard method of selecting proportions and aggregates and by inferior workmanship, due to lack of proper supervision, or lack of judgment, and feeling of responsibility, with the idea that concrete is concrete, which will withstand any usage as good masonry construction. This is a wrong conception of the importance of this class of work. The selection of proper proportions and well-graded aggregates of good quality, coupled with good workmanship, the proper consistency of the mix and the thoroughness of the mixing, depositing, compacting and spading are factors which must be considered and insisted upon if a good, dense, strong and durable concrete is to be obtained.

With such precautions employed, inert fillers or compounds used in the proper proportions, impermeable and good concrete should be obtained.

In presenting results of tests of waterproofing materials added to the ingredients of concrete, the proportions of the mixture are at times stated in two different ways. One method is to state that a certain proportion of waterproofing material was mixed with the cement and then the proportions of the test specimens are given as so much of the cement mixture to aggregate. Other tests are described in which an amount of waterproofing material equivalent to a certain percentage of the cement used is added to the concrete materials. The results of such tests cannot be correctly compared without reducing them to a common ratio between cement and aggregate.

When dry compounds are used from 1 to 2½ per cent of the cement used is recommended by the manufacturers, while for the liquid compounds from 4 to 8 per cent of the amount of water used is recommended by them.

The cost of concrete is increased by the addition of such materials from 80 cts. to $1.2 per cubic yard for dry compounds and from 50 cts. to $1 for the liquid compounds, pe cubic yard of concrete.

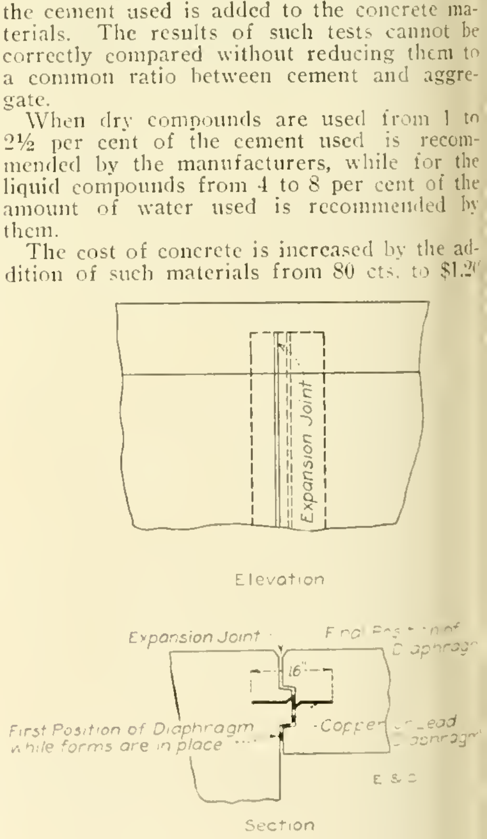

Expansion Joint
Elevation
Expansion Joint
Final Position of Diaphragm
16″
First Position of Diaphragm while forms are in place
Copper or Lead Diaphragm
E & C
Section

Fig. 5. Method of Waterproofing Expansion Joints.

Active Fillers.—Compounds which are adde to the concrete mixture and which react wit certain of the constituents of the cement t form other compounds which will be iner and fill the voids are included in this class In general these materials are soaps an saponifiable oils. Inasmuch as the water proofing effect of these materials depends upon a reaction which may or may not take place, objection has been made to their use.

WATERTIGHT CONCRETE CONSTRUCTION.

The results of laboratory experiments, supplemented by many examples from practice, have shown that watertight concrete can be made without the use of coatings, membranes or integral compounds. It is reasonable to assume that the porosity of concrete in certain cases is due to the fact that it contains small air spaces or voids throughout its mass, which are connected to each other more or less irregularly, and through which water passes, due either to the presence of the hydrostatic head or to capillary attraction. At the time of placing the concrete, some space is occupied by water carrying in suspension fine particles of cement. It is not necessary to assume that continuous capillary passages must be left in the concrete in order that as it dries the water may get out. It is probable that the excess of water passes out of the concrete in drying in such a state as to leave behind no pores through which water could again find access to the interior of the concrete or penetrate the structure.

The question of watertight concrete is then a problem of reducing the size and number of voids. Sands contain voids ranging from about 25 to 40 per cent of the total volume of dry loose sand. The proportions of cement to aggregate required to make a mixture of the maximum density with sands of these extreme values are about 1:1½ to 1:2½. Experience has demonstrated that mortars leaner than this are not suitable for work requiring considerable strength or density, so that the proportions used in ordinary engineering work are sufficiently rich to produce a watertight concrete, provided the aggregates possess the requisite qualities.

Samples of crusher run limestone show 37 per cent voids for each of two specimens one having a maximum size stone passing 2½-in. sieve, the second passing 1¼ in. sieve. A broken stone passing a 2½ in. ring and retained on a ¾-in. screen had 46 per cent voids. Feret found about 52 per cent voids in samples consisting of stones of about one size, for each of three different sizes. A similar variation in the percentage of voids with gradation in sizes of particles is found with gravel, for screened gravel of approximately one size of particles 40 per cent to 45 per cent, for a well-graded gravel containing sand 25 per cent.

The amount of voids in a mixture of aggregate and cement is the least when the cement is just sufficient to fill the voids in the aggregate, since the cement paste itself is less dense than the coarse material of the aggregate.

A slight deficiency in cement produces a porous concrete because the unfilled voids are large enough to permit the passage of water, while properly-made concrete containing an excess of cement, though it may be of lower density than the former, is impermeable after hardening since the voids in the cement paste are too small to permit the passage of water.

Tests have failed to discover a substance which, added to the concrete materials, will increase the density of the concrete, or which fills the interstices between the particles of the aggregates, hence it is not [illegible] that improvement as regards impermeability of concrete containing sufficient cement can be made by the addition of any material to the concrete mixture.

Some engineers apprehend that accurate grading and proportioning according to ideal requirements necessitates extreme care and considerable expense, and therefore reject this method of obtaining watertight construction in favor of the integral compounds, which are in reality based upon the same principle as the grading, [illegible] which are uncertain as regards permanent impermeability and are detrimental to the strength of the concrete.

While it is true that concrete in which the amount of cement used is slightly in excess of the voids in the aggregate, and in which the aggregate is so graded as to contain a minimum amount of voids, is an ideal mixture as regards density and strength, the requirements for watertight concrete do not demand the maintenance of exact proportions of this nature.

Experience has proved that materials, as supplied for large works, run uniformly enough to permit the proportioning and grading to be maintained at such a degree of excellence as to insure watertight construction at a very small expense for testing.

The following abstract from the results of laboratory tests made by the United States Bureau of Standards, Technologic Paper No. 3, are here quoted:

> These tests show that the permeability of concrete was not dependent entirely upon the quantity of cement used in proportion to the total aggregate, but depended also upon the ratio of coarse aggregate to fine aggregate. It will be observed in the case of sand No. 4, that the 1:1½:7½ proportion was decidedly more impermeable than the 1:2:4 proportion, although the former contains considerably less cement in proportion to aggregate.

Tests designed to show the effect of waterproofing materials, especially such as are added as fillers, should present a granulometric analysis of the aggregate, as comparisons are valueless without such information. It is to be expected that tests on mortar in which a sand was used having a deficiency of fine particles would show increased impermeability and increased strength upon the addition of a small amount of fine material. On the other hand, if the aggregate already contains as much fine material as it requires, addition of a fine material as waterproofing may be expected to decrease the strength and have no beneficial effect as a waterproofing material.

The method of proportioning the aggregate by mechanical analysis, which is described by Taylor & Thompson as exact and scientific, is recommended. The granulometric analysis requires a very inexpensive equipment, and a complete analysis of an aggregate may be made in less than one hour's time. By its use definite data may be obtained upon which to base conclusions as to the necessity of and method of improving the concrete mixture.

In discussing the use of exterior coatings as against impermeable construction, the point is often advanced that although there is no doubt that watertight concrete can be made, the watertightness is of no avail when cracks occur in the structure.

The subject of cracking is one of design. Cracks are caused by failure to properly provide for primary stresses to which the structure is subjected, by faulty details, by settlement of foundation, by shrinkage of concrete when hardening in air, and by stresses developed in the concrete due to temperature changes.

Where concrete is to be deposited under circumstances which make it impracticable to construct watertight concrete, a special form of waterproofing should be provided.

Drainage.—The first requisite in designing any structure when water is to be kept out from the interior or from beneath, is to provide means of getting rid of the water as directly and as quickly as possible. Methods of providing drainage differ with the [illegible] of the structure.

During the construction of basements and pits, drainage can be maintained by pumping, and permanent drainage should be provided whenever a free outlet can be obtained.

Drainage of arches and culverts is provided by sloping the extrados to the back of the abutments and to the piers, placing down-spouts at piers and drain pipes behind abutment.

Drainage of retaining walls, abutments and subway walls is provided by one or more lines of drain pipes, placed at different elevations along the back of the wall.

In tunnels the extrados of the arch may be provided with sufficient slope to facilitate the flow of seepage water to the sidewalls. The back filling consists of porous material, which will permit the ready passage of the water. Side-drains and connecting under-drains should be provided.

The drainage of subaqueous tunnels differs from the general problem of drainage, and is not concerned with waterproofing, in that it is a problem of handling water on the inside of the tunnel. This is usually accomplished by pumping from sumps.

The foundations of masonry reservoirs should be drained to insure the stability of the structure.

The solid floors of steel or reinforced concrete bridges may be drained by sloping the finished surface of the floor from the center to each end, and carrying the water away back of the abutments, or the water can be carried away by down-spouts at the intermediate points or supports.

Probably the commonest method of drainage solid-floor bridges is to slope the deck to one abutment or from a summit to both abutments. A continuous waterproofing layer extends over the deck and the top of the abutments and extends down over the back of the abutments to prevent the seepage of water at the bridge seat.

The surface of the waterproofing and its protection must have sufficient grade to carry away surface water. In the case of bridge floors, it is recommended that this grade be not less than 6 ins. in 100 ft. It is customary, when bridges are on sufficient grade, to have the waterproofed surface at the same grade, the water being carried down over the back wall of the lower abutment where drainage is provided by coarse backing and open-joint drains.

An objection to this method of drainage is made by some who find that in the spring, when the surface ice and snow melt and the filling back of the abutments is still in a frozen condition, the water does not escape freely, but accumulates and eventually seeps through at the end of the bridge and flows over the face of the abutment. Another objection is that in bridges having supports at curb lines and in the middle of the street, whether of flat slab construction or of steel troughs filled with concrete, cracks in the waterproof covering and in the concrete filling are likely to appear where joints are not provided over these supports, and where joints are provided, trouble is likely to be experienced in preventing the seepage of water.

When the troughs of steel bridges run transversely to the track and the filling in the troughs is omitted, the individual troughs may be drained through outlets in the bottom of the troughs into a drainage gutter suspended beneath the deck. These gutters may empty into pipes which run through the abutments and empty outside the embankment. Difficulty is found in obtaining a seal between the waterproofing and the drain pipe or opening in the trough.

When the troughs of solid floor steel bridges run parallel with the tracks, the water is usually carried over the abutments as in the concrete floor bridges.

A method sometimes used on solid-floor bridges in which the deck is filled up above the top of the steel with cement or bituminous concrete is to divide the floor of the bridge into rectangular sections each of which is sloped to a drain pipe at one corner which carries the water through a down spout at one of the supporting columns.

Much difficulty has been experienced with all types of waterproofing on steel bridges in preventing the leakage of water along the webs of girders. Although the concrete filling of the deck may be carried up above the top of the rail and great pains may be taken in [illegible] with a waterproofing material between the girder web and the concrete, leaks usually develop along the girder.

Several bridges have been built in which a [illegible] angle or Z-bar extending the [illegible] has been riveted to the inside of the girder to prevent the flow of water down the web of the girder. By carrying the concrete [illegible] underneath the outstanding leg of the flashing angle or Z-bar an efficient flashing is obtained. Good results have been obtained in the case of through girder bridges by carrying the concrete filling up under the top flange of the girder.

In considering the conclusions presented in Bulletin 64 in regard to reinforcing over supports, the following remarks of President Armstrong, of the Western Society of Engineers, are of interest:

> In large railroad structures it is impracticable to reinforce concrete so that there will be no cracks over a line of supports; good engineering would not permit such practice. It would be better to allow the concrete to crack or to leave a joint there, and then provide some means of keeping out the water. In the lighter structures, it is practicable to reinforce the concrete so that the reinforcement will prevent cracks at supports.

A joint in the waterproofing which will allow of movement of the ends of adjacent spans at supports is believed to be necessary. The use of a metal flashing between concrete slabs over joints has been used.

When the steel troughs run transversely to the track, a slight movement under traffic is to be expected at the connection of the troughs to the girders. Consequently, it would seem necessary to keep the water away from these connections by means of flashing and providing sufficient slope toward the center of the floor, adjacent to the girders.

Figures show methods of waterproofing various structures.

CONCLUSIONS.

(1) Watertight concrete may be obtained by proper design, reinforcing the concrete against cracks due to expansion and contraction, using the proper proportions of cement and graded aggregates to secure the filling of voids and employing proper workmanship and close supervision.

(2) Membrane waterproofing, of either asphalt or pure coal-tar pitch in connection with felts and burlaps, with proper number of layers, good materials and workmanship and good working conditions, is recommended as good practice for waterproofing masonry, concrete and bridge floors.

(3) Permanent and direct drainage of bridge floors is essential to secure good results in waterproofing.

(4) Integral methods of waterproofing concrete have given some good results. Special care is required to properly proportion the concrete, mix thoroughly and deposit properly so as to have the void-filling compounds do the required duty; if this is neglected, the value of the compounds is lost and their waterproofing effect destroyed. Careful tests should be made to ascertain the proper proportions and effectiveness of such compounds.

Integral compounds should be used with caution, ascertaining their chemical action on the concrete as well as their effect on its strength; as a general rule, integral compounds are not recommended, since the same results as to watertightness can be obtained by adding a small percentage of cement and properly grading the aggregate.

(5) Surface coatings, such as cement mortar, asphalt or bituminous mastic, if properly applied to masonry reinforced against cracks produced by settlement, expansion and contraction, may be successfully used for waterproofing arches, abutments, retaining walls, reservoirs and similar structures; for important work under high pressure of water these cannot be recommended for all conditions.

(6) Surface brush coating, such as oil paints and varnishes, are not considered reliable or lasting for waterproofing of masonry.

Water Works

The Necessity for State Supervision of Water Works Operation.

This journal, in common with others in its field, has frequently emphasized the desirability of state supervision of the design, construction and operation of engineering works. In previous issues we have called attention, particularly, to the need of such censorship in connection with the design of dams and the operation of sewage treatment plants. The present article takes up the same general subject with special reference to water works operation. The discussion and data following are reprinted from the paper before the Illinois Water Supply Association by Allan J. McLaughlin, Surgeon, United States Public Health Service, and director of the field work of the International Joint Commission.

State supervision of water works plants should not be a nominal duty performed in a perfunctory manner. It should be a very close supervision of the operation of the plant.

In many states there is a lack of law giving the necessary specific authority to the State to control water supplies and even when such law exists in some instances the State officials depend upon the local authorities for results, with few inspections and very little actual control of the water supplies.

Obviously, in large cities like New York, Chicago, or Philadelphia, which are rich enough to employ efficient operators for their plants and competent bacteriologists to make daily analyses, there is no great necessity for close state supervision, but even here the inspection by state authorities has a salutary effect in keeping up the maximum efficiency. It is in the smaller cities and towns, however, that state supervision and control will have its greatest effect. In order to show the crying need of state inspection and control of water plants, certain instances which came under my personal observation may be cited.

The first thought in municipalities when they get away from the "old oaken bucket" is to secure a supply which shall be adequate in quantity. The next public demand is probably for a clear water if the water is taken from a muddy river. It is only in the last few years that the demand for safe water has been made the paramount issue in public water supplies.

Improvement in water supplies which were grossly polluted with sewage has been effected frequently to eliminate the taste of manufacturing wastes. Gradually municipal officials are beginning to recognize the insistent public demand for safe water. Even now, however, with the lessons of our past epidemics before us it is a difficult task to convince many municipal officials that money should be spent to purify a polluted water supply which is adequate in quantity and reasonably clear in appearance. It is obvious that the water board or water works superintendent must consider the economic side of this question but they should be just as much interested in furnishing a safe water every day in the year as in the heat efficiency developed from their fuel.

The greatest delusion in water supply history is that the Great Lakes water needs no purification. Practically without exception the cities taking water from the Great Lakes should install purification plants to protect their citizens, yet many of them persist in furnishing polluted water without treatment of any kind. It is not alone in the use of untreated water that disaster occurs. Many cities have plants which are structurally imperfect or inefficiently operated.

Ashland, Wisconsin, is a notorious instance of a small city with a slow sand filter plant and a typhoid fever rate of 315 per 100,000 in 1910. In spite of their lesson the typhoid rate since 1910 has averaged 72 deaths per 100,000. Inquiry as to the improvement following the calamity which befell this city in 1910, elicited the following from the Health Officer:

> Your letter wanting to know of the changes in our water works system is at hand. Since 1910 a new filter has been installed. This was in the winter of 1911-1912. In the winter of 1912-1913 the intake pipe was taken up and moved and it was shortened about 900 ft. In the early winter of 1912 a chlorine plant was installed and they have been using it ever since except when the taste and smell of the water became so noticeable, then it would be cut out for a few days until the taste and smell had disappeared.
>
> These are all the changes that have been made that I know of and if there is anything more that I can help you with or anything that you want to know that needs looking into, I will be very glad to do it for you.

The "new filter plant" said to have been installed in the winter of 1911 and 1912 did not prevent the winter and spring outbreak January to April, 1913. The "chlorine plant" which is said to have been in use since 1912, "except when the taste and smell became so noticeable," seems to have been equally ineffective in reducing Ashland's typhoid fever rate.

The idea seems to be prevalent that when a public water supply is finally conceded to be grossly polluted that the installation of a filter plant settles the problem. It is supposed to be a miraculous fool-proof mechanism which will operate itself even while the operator sleeps or goes to dinner. I know several excellent gravity mechanical filter plants, and perhaps this type more nearly approaches the fool-proof than any other which are operated by a kind of a promoted coal-passer or stoker. The only attempt at bacteriologic control is the sending of a sample to the state capital or the state university once or twice a year.

One plant in a small New York town was inspected by one of my Canadian colleagues. It was a mechanical filter plant with alum as a coagulant. The man in charge was using no alum. He said it looked to him as if the alum was unnecessary as the water was pretty good anyway.

Perhaps the least fool-proof of any of the purification processes is the administration of hypochlorite of lime. The dosage of the chemical to be effective without imparting offensive taste to the water requires very careful and skillful attention. Yet commonly there has been an effort to dose the water by set rule of so many pounds per million gallons regardless of the flucuations in the raw water. The result is that in order to avoid taste in times of low organic content the dose is fixed too low to be effective when the pollution and organic matter increases at certain seasons due to floods or other causes. Cleveland installed hypochlorite and in 1912 the first year of operation had the lowest typhoid rate in the history of the city. In 1913, the tendency to tamper with and cut down the dose became more mainfest and there was a sharp rise in typhoid in Cleveland following the spring freshet in the Cuyahoga River. In one of the smaller Michigan cities I asked the stoker in charge how much "hypo" he was using. He said they had been using about 6 lbs. per 1,000,000 gals. but the mayor telephoned him that he could taste the "hypo" and to cut it down. He said he guessed he was using about 4 lbs. per 1,000,000 gals. now.

It is to correct these defects that state control is necessary. Those states which have no law covering this should lose no time in securing such a law. Where sufficient law exists the function of supervision and control should be exercised by the state with frequent inspections and definite rules and regulations covering the needs of each municipality. Defects in construction or operation could be thus corrected and efficient maintenance and operation secured. Daily bacteriologic control is a necessity and should if possible be carried out in the plant itself.

The greatest obstacle to proper operation nd control of plants has been the difficulty f securing the right man to place in charge f the plant. The best type of man for this osition is a graduate in sanitary engineering. le will not only be conversant with the mechanical details of the plant, but will be able o adjust his chemicals according to the constituents and needs of the raw water. Most nportant of all, he will be able to make daily acteriological examinations to determine the fficiency of purification. Nearly all the dissters due to sewage polluted water supplies which have occurred were due to lack of daily acteriological knowledge of the public suply or the inefficient operation of plants by nskilled men. Personally, I believe the employment of such a graduate is economy even n small cities. I can conceive, however, of ases where it is impossible for economic reaons to pay the necessary salary. In these ases local men must be employed, and trained o do the work. Here the State Board of Health, or as in Illinois the State Water Surey, will find a very useful function. The tate authorities could supervise the installaion of a small, inexpensive laboratory equipment in small plants and give instruction to he local man in making the necessary water xaminations. Whenever possible, however, oung graduates of sanitary engineering chools should be employed, and such men re well worth their salary considering the aving in the economical adjustment of chemials and fuel costs made possible by intelliint supervision. The greatest asset to be redited to skilled operation is the saving of human life effected and the satisfaction of nowing that safe water is being furnished every day.

p. m. Considerable trouble has been found at various times with the growth of crenothrix in the mains and at other places, and the dripping of water gave a stain to porcelain and other material. The collection of sediment in mains at dead ends and in connected lines having little movement of water has been such as to give trouble when a current was formed by the opening of fire hydrants or by other large drafts of water and the sediment was stirred up and carried through the service pipes. The small amount of iron in the water would not be so troublesome, if it were distributed through the supply in the way it comes from the wells, but when a large amount of iron and the accompanying growths are concentrated at particular times and in special localities in the city the difficulties are emphasized. Under these circumstances the company decided to construct an iron removal plant along lines which follow the methods used in an experimental plant at this place. The iron removal plant finally built for this purpose is here described from information taken from the paper by Prof. A. N. Talbot, of the University of Illinois, who designed the plant, before the annual convention of the Illinois Water Supply Association.

Design, Construction and Operation of the Iron Removal Plant of the Champaign and Urbana (Ill.) Water Company.

The Champaign and Urbana Water Co. obtains its supply from wells 140 to 200 ft deep hich draw their water from fine sand in the rift. This water contains about 2 parts of on per 1,000,000. As pumped from the wells, he water holds about 40 parts of carbon dioxle per 1,000,000, and practically no dissolved xygen. The aeration obtained as the water discharged from the wells into the resernir adds oxygen to an amount of about 4 p.

The process used involves aeration and filtration through a sand filter, no coagulant being applied. The filter is much like the ordinary rapid sand filter except that coarser sand is used. Aside from removing the iron from the water, among the special features of this plant are the method of measuring the raw water as it is discharged into the filters, and the provision for variable level of water in the filter as a means of regulating the rate of flow through the filter.

The water is pumped from the wells into a receiving reservoir, which will be termed the raw water reservoir. Some aeration is obtained as the water is discharged into the reservoir. The water is pumped from the raw water reservoir upon the filters, and the filters discharge by gravity into the clear water reservoir. The two reservoirs named have been in use for many years.

The filter consists of four units of a nominal capacity of 500,000 gals. each per 24 hours, located in such a way that the number of units may be increased when needed. Fig. 1 shows the plan of the filter and piping system. Fig. 2 is a transverse elevation of the filter house. Each filter is 12 by 15 ft. in dimension, giving, with the space taken out at the corners, a cross-section of 178 sq. ft. The filter walls extend 9 ft. above the floor of the filter. The walls are built of concrete reinforced with steel rods, but their thickness is probably sufficient to resist the head of water without the reinforcement. The walls and floor are of 1:2:4 concrete and the foundations of 1:3:6 concrete. A fill of concrete made of one part cement, four parts of sand, and eight parts of cinders was placed under part of the floor. The foundation was first put in to the height of the middle of the depth of the floor. The walls were then built, the north two filters being constructed first and the south two filters next, the work in each case being completed in a working day. A thickness of floor was next put in up to the level of the strainer system and after the strainer system was in place the floor was completed.

The strainer system and air system were furnished by the American Water Softener Co. of Philadelphia. The main collector is a cast-iron pipe 10 ins. in diameter which reduces to 8 ins. in the wall of the filter. This size gives a velocity of 5.7 ft. per second with a wash of 1,400 gals. per minute, equivalent to a rise of one vertical foot per minute. Two-inch tees are connected with this main collector and branch into 2-in. extra heavy wrought iron pipe in opposite directions. Hodkinson brass strainers with a throat opening 0.25 in. in diameter are used. The spacing of the strainers is 6 ins. center to center in each direction. Fig. 3 shows the loss of head through these strainers for a throat diameter of 0.23 in. for different rates of discharge.

A separate air system is used. The main is 1-in. wrought iron pipe which branches to 3 in. in two directions at the level of the gravel. One-half inch brass tees and brass pipe feed from these branches. The holes are 1/16-in. in diameter spaced 3 ins. apart and placed at 15 points. The drilling of the brass pipe is shown in Fig. 2.

Each filter has two wash troughs which rest upon supports set into the end walls. The wash troughs have for the bottom a 10-in. 15-lb. steel channel, for the sides ¼-in. steel plate, and have 2x1½-in. angles at the top set ½ in. below the edge of the plate. They are tied across by angle braces at five points in their length. The wash troughs are 10 ins. deep at the upper end and 11 ins. deep at the lower end. These wash troughs have proved

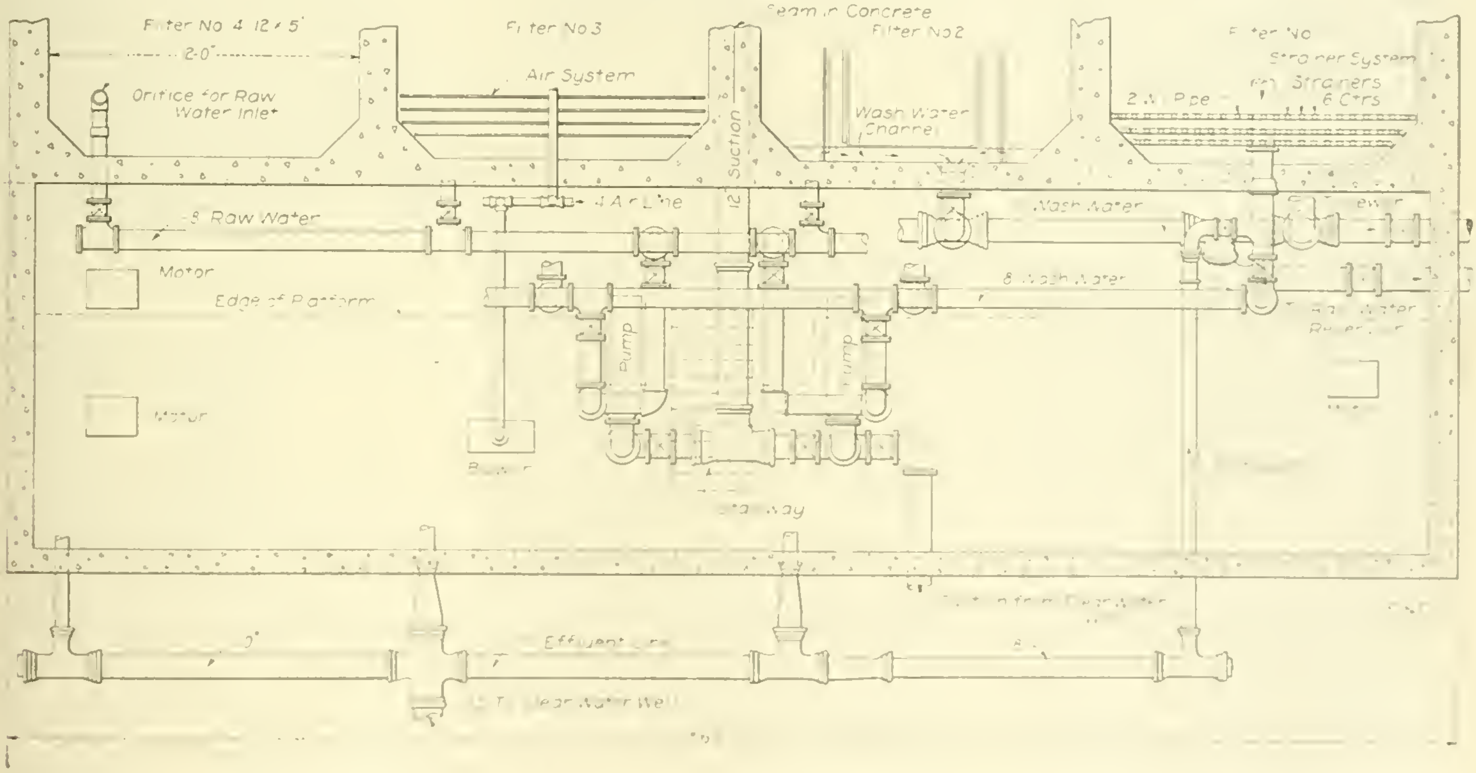


Fig. 1. Plan of Filters and Piping of Champaign-Urbana Iron Removal Plant.

to be very satisfactory. The two wash troughs discharge into a channel placed along the ledge in the front wall of the filter, the wall being recessed 2 ins. to give a larger cross-section, and the channel slopes both ways to an opening in the wall which connects with the 10-in. discharge line. This 10-in. drain connects with the sewer and there is a by-pass which permits a discharge into the raw water reservoir.

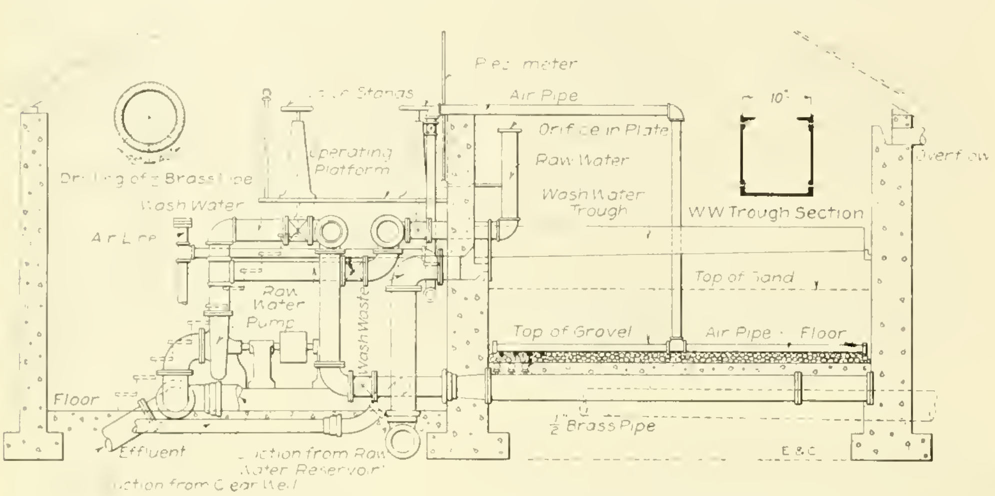


Fig. 2. Section Through Filter House of Champaign-Urbana Iron Removal Plant.

The raw water is discharged into the filter through an orifice in the vertical pipe, rising as a jet and dropping vertically to the surface of the water in the filter. The result is an increased aeration and the water is practically saturated with oxygen when it reaches the filter. Fig. 4 shows the arrangement of the orifice. The vertical pipe shown has a connection with the glass tube. A scale along the tube properly graduated indicates the quantity of water which is being discharged, and the raw water valve is regulated until the desired rate of filtration is indicated on this scale. The apparatus forms a simple, accurate and sensitive method of determining the quantity of water discharged upon the filter. Figure 4 also gives the discharge of the orifice at different heads

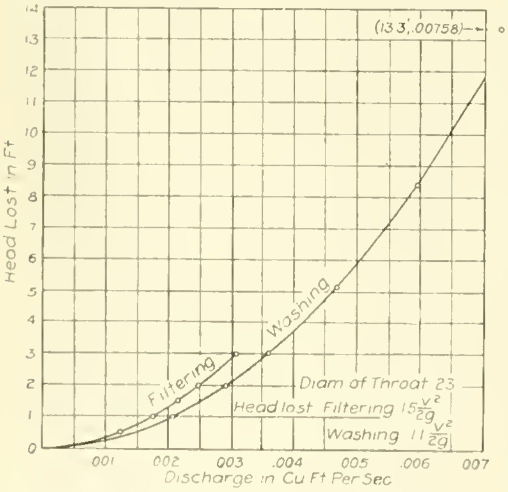


Fig. 3. Diagram Showing Loss of Head in Hodkinson Filter Strainer With Different Rates of Filtering and Washing.

Filtering 125,000,000 gals. per acre per day is equivalent to 0.00111 cu. ft. per second per strainer when placed 6 ins. center to center in each direction. Rise of wash water 1 ft. per minute is equivalent to 0.00416 cu. ft. per second per strainer when placed 6 ins. center to center.

The filter bed consists of about 30 ins. of sand and 8 ins. of gravel. The gravel ranges between ¼ and ½ in. The sand is washed bank sand from Covington, Ind. After putting it in place in the filter, the sand was thoroughly washed several times by the usual filter washing process, and the layers of fine particles which formed on the surface were then removed. Although this sand is not as uniform in size as is the filter sand which is usually specified, the saving in cost was so much that it was thought best to use it.

Two 8-in. American centrifugal pumps are used for pumping the raw water and wash water, being installed in such a way that either pump may be used for the raw water or the wash water. The pumps are run by 15 H.P. induction motors (with wound rotor), belt driven. Drum controllers and rheostats permit the speed of the pump to be changed 20 per cent or more. This arrangement gives conditions of variable speed adequate for the range of discharge desired, and has proved a very satisfactory one. Care was taken in advance to estimate carefully the head pumped against and the speed necessary for the different rates of discharge. The blower was furnished by the Connorsville Blower Co. and has a capacity of 360 cu. ft. of air per minute.

The operating platform runs along the front of the filters at a height of about 8 ft. above the floor of the building. The various valves for the several filters occupy the same places relatively and are operated from this platform. All the principal valves of the piping system except those on the suction lines are also operated from this platform, as are the motors. The east, north, and south walls of the filters form part of the enclosing building. The remainder of the walls of the building are brick covered with stucco.

OPERATION.

As ordinarily operated, three filters have been in service and the normal amount of water used has been about 1,500,000 gals. per day. After a filter is washed the raw water discharge valve is set to give a discharge of, say, 350 gals. per minute. This is done when the water over the filter stands at a medium level, say 2 ft. below the overflow. The effluent valve is set so as to show the amount of lost head which experience tells is the usual amount just after the washing of the filters. If the head of water on the filter at starting is more than sufficient to produce the desired rate of filtration, the level of the water lowers, and if it is not sufficient to give the desired rate the level rises, until the level adjusts itself to give a filtration rate through the filter equal to the discharge upon the filter. As the the filter clogs, the level of the water in the filter will rise to give the necessary additional head. A range of 4 ft. in the level of the water is available. A change in the effluent valve two or three times between periods of washing is sufficient to keep well within this range and to prevent overflow from the filter to the raw water reservoir. It has been found that the regulation of the rate of filtration is very easily managed, and the method has proved to be satisfactory. It has advantages over the use of a controller on the effluent pipe whose operation is hidden and uncertain, and the amount of lost head is less with this device and the filter need not be raised to provide for such lost head. It also gives an accurate measurement of the water discharged upon each filter, which the ordinary controller does not do. The pumping arrangements work very smoothly and the piping system has proved convenient and satisfactory.

When the filters were first put in operation and while the sand was new, the filtration results were fully satisfactory. Only a trace of iron was found in the effluent. The runs were 30 hours or more and washing of the filter seemed to remove all sediment and restore the sand to its first condition, and no higher losses of head at starting were found than at the beginning of operation. After a month or six weeks of service, however, there was such an accumulation of sticky material in the filter bed and the sand grains had become so coated with this sticky substance, that adequate washing over the whole area of a bed was impracticable, and the loss of head through the filter immediately after the bed had been washed became very high. The intervals between necessary washing was reduced to a few hours. While the filters were in this condition, large cracks would appear over the surface of the filter running well down into the sand, the appearance being much the same as that of mud cracks after drying. The effluent became poorer and poorer until at one time not half of the iron was removed by the process. To remedy this condition the sand from a filter was removed and given a thorough washing, and returned to its place in the filter. The result was that the operation of the filter became as satisfactory as when it was first put in operation. The other filters were then given the same treatment. As showing the condition of the sand layer, it may be said that pumping the sand through a centrifugal pump cleans it and separates the sticky material from the sand satisfactorily, and that the foreign matter will then float off and drain away. Further work is required to perfect the process of this occasional thorough washing of the sand. It is thought that the sand of a filter may be washed by running it through a centrifugal pump, and returned to its place directly, without using much more water than is required for the ordinary daily wash of the filter. The sand of the bed has been washed twice in eight months.

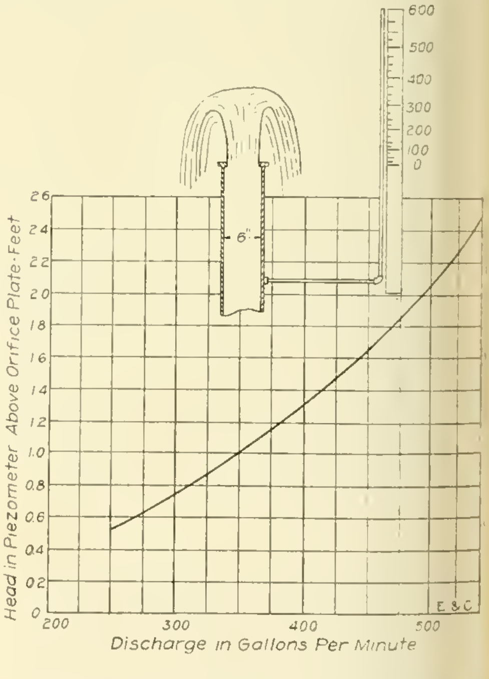


Fig. 4. Diagram Showing Discharge from Orifice with Different Piezometer Readings, and Arrangement of Aerating Supply Pipe and Piezometer at Champaign-Urbana Iron Removal Plant.

At present the occasional use of a fire hose n the bed in stirring and washing the sand rains has been found to be effective, and this ill extend the length of the interval between he times when it will be necessary to scour he sand. The amount of water so used is ot more than that required for the ordinary aily wash.

Changes have been made in the practice in he usual washing of the filter. The air wash used as a means of stirring and is not generally put on while the wash water is being asted. The bed is stirred manually by means f a garden rake. Two washes are given each ay, as that has been found to give better results and not to increase the amount of wash ater required.

During this time, experimental work has een carried on through the co-operation of he State Water Survey, with one or more f the filters to learn if the application of hemicals to the water would lessen the difficulties in the washing of the filters. Relave to the use of these chemicals suffice it to ay that no advantage was found by the aplication of any of the chemicals used.

This clogging of the filters was the only ifficulty which had been feared and now that his obstacle in the operation of the filter has een overcome the results are entirely satisfactory. The removal of iron from the water ithout the use of chemicals is found to be very simple process. It seems probable that he same process is applicable to many of our rift well waters.

The entire work of constructing the filters as done by the water company's force and y day labor, no contracts being let. Mr. F. . Amsbary, manager of the water works ompany, was in general charge of the work. Ir. Marquard, engineer of the water company, superintended the installation of the iping, pumping, and strainer systems and of he filter materials. Mr. G. C. Habermeyer, ade details of the plans and inspected the onstruction for the writer.

Duty Test Data of the New Triple Expansion Pumping Engine of the Springfield, Ill., Water Works.

The city of Springfield, Ill., in the year of 911 commenced the reconstruction of its mping station at a cost of approximately 5,000, and also contracted for a pumping gine at a cost of $52,369. These improveents have all been made under the direction W. J. Spaulding, commissioner of public operty.

The pumping engine is of the Holly vertical ple expansion crank and flywheel type of a rmal capacity of 10,000,000 gals. per 24 ours. The engine was erected by the Holly anufacturing Co. at the city's pumping staon four miles north of the city on the banks the Sangamon River. As a check on the aranteed duty of this pump an efficiency t was made directly following its installaon. The test data observed are here given presented by Mr. Walter Reid, superintendnt of the Springfield water department in s recent paper before the Illinois Water pply Association.

The dimensions of the cylinders are: High essure, 26 ins.; intermediate, 48 ins.; low essure, 72 ins.; stroke, 42 ins. All of the am valves are of the Corliss type, also the haust valves of the high pressure and intermediate, while the low pressure are of the ppet type.

The diameter of the steam pipe is 16 ins.; haust pipe, 20 ins.; suction, 30 ins.; discharge pipe, 24 ins. There are three water ungers 23 ins. in diameter, 42 ins. stroke. he plungers are outside packed and are dictly under the steam cylinders. The pump d contains 1,050 rubber valves 3½ ins. in ameter, and are so located that they are adily accessible. The condenser is of the rface type and is located in the suction pe, and has 1,000 sq. ft. of cooling surface.

The duty guaranteed by the builder per 1,000 lbs. of dry steam consumed by engines, its jackets and attached auxiliaries at normal capacity and pressure (277 ft.) was 170,000,000 ft.-lbs.; duty three-quarter capacity, 165,000,000 ft.-lbs.; duty one-half capacity, 160,000,000 ft.-lbs. The normal capacity rated at 7,000 gals. per minute based on plunger displacement and a total head of 277 ft., maximum 375 ft. Steam pressure at throttle, 150 lbs., 32 R.P.M. which equals a piston speed of 221 ft. per minute.

On Jan. 21, 1914, the necessary gages were connected to the discharge and suction pipes, tanks for collecting, and scales for weighing the water from the jackets, receivers, and discharged from condenser were all placed and tested and corrected previous to the final test. A number of preliminary tests were run and adjustments of steam valves for the most efficient distribution of steam and equalization of load for each cylinder was made.

The actual test at full speed during twelve hours was as follows:

TEST AT FULL SPEED.

Steam pressure at throttle, 150.5 lbs. per square inch.

Vacuum in exhaust pipe, 27½ ins.

Average revolutions per hour, 1959 (or 32.5 R.P.M.)

Average discharge pressure, 118.5 lbs. per square inch (273.35 ft.)

Average suction lift, 12 ft.

Difference in elevation of gages, 2.5 ft.

Total water pressure, 287.85 ft.

Average total steam consumption per hour, 6,344 lbs.

Condensation from jackets and receivers, 13.15 per cent.

Moisture in steam, 2.28 per cent (145 lbs.)

Total dry steam per hour, 6,199 lbs.

Duty per 1,000 lbs. dry steam, 171,830,000 ft-lbs. (I.H.P. 459.6.)

TEST FOR THREE-QUARTER CAPACITY.

Duration of test, 4 hours.

Steam pressure, 149.8 lbs. per square inch.

Vacuum in exhaust pipe, 27¼ ins.

Average revolutions per hour, 1,461 (or 24.1 R.P.M.)

Average discharge pressure, 117.2 lbs. per square inch (271 ft.)

Suction lift, 11 ft.

Difference in elevation of gages, 2.5 ft.

Total water pressure, 281.5 ft.

Average steam consumption per hour, 4,776 lbs.

Moisture in steam, 2.8 per cent (131 lbs.)

Condensation from jackets and receiver, 12.75 per cent.

Average dry steam per hour, 4,642 lbs.

Duty per 1,000 lbs. dry steam, 169,256,000 ft.-lbs. (I.H.P. 346.5.)

DUTY FOR ONE-HALF SPEED TEST.

Duration of test, 4 hours.

Steam pressure, 150.3 lbs. per square inch.

Vacuum in exhaust pipe, 27¼ ins.

Average revolutions per hour, 1,068 (or 16.1 R.P.M.)

Average discharge pressure, 116.1 lbs. per square inch (269 ft.)

Average suction lift, 10.5 ft.

Difference in elevation of gages, 2.5 ft.

Average total water pressure, 282 ft.

Average steam consumption per hour, 3,[illegible] lbs.

Moisture in steam, 2.31 per cent (83.7 lbs.)

Condensation from jackets and receivers, 12.5 per cent.

Average steam corrected for moisture, 3,196.3 lbs.

Duty for 1,000 lbs. dry steam, 167,810,000 ft.-lbs. (I.H.P. 282.5).

In order to insure accurate results readings were taken of the water pressure every ten minutes from a test gage on the force main. Readings were then reduced to hourly averages in order to check the performance of each consecutive hour while the test was being made. Indicator diagrams were taken every hour from each of the steam cylinders in order to show the steam distribution, the speed being kept constant by a fly ball governor attached to the engine.

We have in our plant in addition to the Holly triple pump an old Worthington triple pump, 18 in. x 29 ins. x 46 ins. x 24 in.—26½ in. x 24 in., of 8,000,000 gals. capacity and a compound pump, 21 ins. x 43 ins. x 36 ins.—20 ins. x 36 ins., of 4,000,000 gals. capacity. At the conclusion of the pumping test we ran a comparative fuel test. The new Holly triple pumping engine consumed 3,900 lbs. of coal to pump 780,000 gals. of water, which is equivalent to 1 lb. of coal per 200 gals. of water pumped. The Worthington triple, 6,105 lbs. of coal per 800,000 gals. of water, which is equivalent to 1 lb. of coal to 131 gals. of water. The compound pump, 7,800 lbs. of coal per 680,000 gals. of water, which is equivalent to 1 lb. of coal to 87.7 gals. of water. Screenings was the fuel used for these tests.

The difference between 200, 131, and 87.7 gals. of water pumped per 1 lb. of coal indicates the economy realized. All of the boiler room auxiliaries were in use during the coal test.

Economic and Service Advantages of Pumping Public Water Supplies by Electricity.

In many of the smaller cities throughout the country water supply systems would be installed if the cost of operation could be brought within the means of the community. Even in many of the larger cities where systems are now in operation there is room for distinct improvement in the operating costs. For the steam driven pumping station the large items in the operation expenses are those for fuel and attendance. Both of these items are either absent or lowered materially in the electrically driven station. Mr. J. M. Bryant, assistant professor of electrical engineering at the University of Illinois, recently read a paper before the Illinois Water Supply Association on the economic and service advantages of pumping city water by electricity which we reprint herewith. Prof. Bryant's paper also contains helpful suggestions on the selection of suitable equipment for this service.

Nearly all of these small communities of the class above mentioned are within easy reach of a transmission system or have one or more electrical plants in their midst. Power can be produced in one large power plant located at a water power privilege or within easy reach of a coal supply than in several smaller stations to some of which coal may have to be hauled by wagons. The problem of electrifying a pumping plant becomes an analysis of the following items:

1. First cost of the equipment.
2. Cost of operation including power, repairs and labor.
3. Reliability of the equipment.

In this discussion of the question it will be assumed that electric power is available for driving the pumps, and that this may be obtained at a fair rate since the pumping company or the city has recourse to the Public Utilities Commission (in several states) at any time for an adjustment of rates.

Cost of the Pumping Units.—A comparison of the cost for steam and electrically driven pumps of the same capacity reveals the fact that the electric equipment costs from one-fourth to one-half that of the steam driven units. Considering that, for new installations, a saving is also made in the first cost for boilers, buildings, etc., it certainly is fair to assume that the cost of the power plant portion of the water works equipment is reduced to one-fourth that for steam-driven stations.

In order to analyze the effect of this reduction on the annual fixed charge the data given in Table I are taken from an average of 22 cities in Wisconsin from the reports of the Wisconsin Railway Commission. (7 W. R. C. R. 301, 8 W. R. C. R. 311.)

Adding together items 5 and 6 of Table I it may be seen that the total cost in power plant equipment and buildings represents 15.23 per cent of the total cost new of the equipment. Lowering this to one-fourth of its present value by the use of electrically driven machinery will effect an annual saving in the fixed charge, which may be taken as 15 per cent of the cost new, by $0.1523 \times 0.75 = 0.114$, or 11.4 per cent. This will in turn affect the cost of operation by $0.15 \times 0.114 = 0.017$, or 1.7 per cent of the present cost of operation of the steam driven station. This means that, for the conditions above assumed, the fixed charges necessary to cover the initial investment is reduced to 1.7 per cent of that investment, or for the same income, and with other operating expenses constant, an increase in the annual profit of 1.7 per cent of the investment. Other items in the operation cost will be discussed under the next heading.

TABLE I.—AVERAGE COST NEW OF WATER WORKS SYSTEM IN 22 WISCONSIN CITIES.

		Per cent of total cost new.
1.	Land	3.10
2.	Wells, intakes and suctions	8.39
3.	Filters, reservoirs, standpipes	6.28
4.	Distribution system	63.75
5.	Power plant equipment	8.86
6.	Buildings and miscellaneous structures	6.37
7.	Office furniture, appliances, tools, etc.	0.80

Cost of Operation.—Items in the annual cost of operation which are less in the motor driven station than for either steam or gasoline driven pumps are the following:

1. Interest on the investment on account of lower investment as explained under the last section.

2. Depreciation is less for the centrifugal pump and electric motor. Depreciation in the motor is almost negligible if the motor is properly housed and cared for. The motor may have to be laid aside on account of obsolescence due to improvements in electrical equipment which would secure additional saving in cost of operation at a later date rather than on account of the wearing out of any of the motor parts.

3. Cost of motor repairs is negligible in comparison with those for a steam or gasoline driven unit on account of the absence of steam and heated gases and of reciprocating parts.

4. Cost of motor attendance may be made very small. If the pumps can be located near the plant which furnishes the power no attendance need be provided outside of that which the power company is willing to furnish for the privilege of cutting off the demand of the large motors at the time of their station peak loads. For large stations and for isolated plants one attendant can care for a large number of motors and pumps. Nearly all the attention that is required is that necessary to start and stop the motors at the proper time and to stand by in case of fire to connect in enough pumps to supply the necessary volume and pressure.

5. Practically no cost for lubricating oils since the pumps and motors are nearly always self lubricating, using the same oil over many times.

6. Very low maintenance cost compared with a steam or gasoline driven plant. The fires in a steam plant have to be kept banked at times of no load and ready to take on fire pumps if necessary. This item of fuel cost for banked fires forms one of the large items in the annual operation sheet of small plants. Maintenance for the motors and pumps is also a much lower item than for steam and gasoline driven pumps.

Reliability.—Reliability in city water pumping is of the utmost importance. In many installations, even in large cities, this factor has been sacrificed in order to lower the first cost of the equipment. There may result from this neglect low pressure in the water mains, and lack of volume for fighting fires causing excessive fire losses and also high insurance rates on real estate for such cities. The electrical motor is more reliable than the steam or gasoline engine. In order to increase the reliability of the whole system the following suggestions are given.

1. The motors should be located above all possible flood levels. Although a few cases are known of motor driven mine pumps operating under water and pumping the mine dry the insulation is not usually guaranteed to stand such treatment.

2. Instead of installing one large motor and pump to furnish the total amount of water for the high pressure service at least two units and preferably three should be installed. When installing three units the combined capacity of any two should be able to supply the peak load of the station. This method of design allows the unit or units in operation to run without throttling and at their full capacity. Throttling a centrifugal pump or running it below speed lowers the efficiency of the equipment and raises the cost of power by requiring a greater number of kilowatt-hours to pump the same amount of water.

When the pumping-station is situated at some distance from the power plant duplicate pole lines should be installed and over routes widely separated. When the wires pass through city streets a fire in a given quarter may destroy one pole line but service may be maintained over the other line. Repairs may also be made on either line and at any time without interrupting the operation of the station.

4. Proper lightning protection should be provided at both ends of the transmission line to the pumping station, to protect all electrical apparatus. The electrolytic arrester has been found to be the most reliable type now in use. Its first cost is slightly higher than that of other forms, but it gives better protection in all cases of lightning or high voltage disturbance from other causes. It operates on the principle of a safety valve, discharging excess electrical pressure from the line and then stopping the discharge at normal pressure, automatically setting itself for the next discharge.

Choice of Motor.—The electric motor can be adapted to drive any type of pump. The greatest efficiency of mechanical transmission is of course secured by the least amount of gearing or belting. The centrifugal pump lends itself best for motor drive since it runs most efficiently at the higher speeds. The higher the speed, the lower the cost of the motor and pump. Information in regard to the proper choice of speeds can always be obtained from the pump and motor manufacturers.

The squirrel cage induction motor is a little more simple and reliable than the wound rotor types. It is suitable for starting centrifugal pumps of small capacities without too great a starting current by the addition of a compensator if the pump outlet is closed when starting. The wound rotor motor with slip rings and external resistance is preferable for sizes above 50 HP. in order to secure the higher starting torque with low line current. Resistance in the primary or line side of the motor for starting should not be used, since the motor then draws from two to five times full load current from the line at the start. This is avoided by the use of the compensator starter or the slip ring motor.

When large pumps are to be driven at a considerable distance from the power plant, transmission line losses may be reduced by the use of one or more synchronous motors in the pumping station. These motors can now be secured with sufficient starting torque for this purpose. By the proper use of synchronous motors a large saving may sometimes be made in power consumption.

Cost of Power.—Electrical power for pumping should receive the lowest rate of nearly all classes of service furnished by a power plant. With water storage facilities in reservoirs and standpipes capable of taking care of the peaks in the pumping demand and also supplying the water for the two or three hours in the day when the commercial load on the station is the greatest, the power for the pumps may be supplied without any increase in the station maximum demand. The increase in the station output incident to this load will then improve the load factor, lowering the cost of producing unit power and delivering it to the switchboard. The proper rate to be charged in any community should be properly determined by a competent engineer or by the Public Utilities Commission, and not left to the governing body of the community and the public utility without expert advice. The public utility should be encouraged to seek the pumping load and the city to co-operate in securing a price giving justice to all consumers. This co-operation should extend not only to the matter of securing rates, but also in installing the equipment best for all concerned and of operating it at times when the company can best afford to supply the power. This same rule should also apply to combined municipally owned plants; each part of the service should bear its proper portion of the fixed and variable charges.

Bridges

Design of the 531-Ft. Truss Spans of the North Side Point Bridge, Pittsburgh, Pa.

(Staff Article.)

The North Side Point Bridge spans the Alleghany River at its mouth, and connects the old city of Pittsburgh with that part known as the North Side (formerly the city of Allegheny). The bridge consists of two main spans of 531 ft. each; a north approach, consisting of about 424 ft. of arch spans of varying length and about 441 ft. of reinforced concrete retaining wall construction on a 4.94 per cent grade; and a south approach, consisting of about 659 ft. of reinforced concrete retaining wall construction on a 5.02 per cent grade and about 254 ft. of reinforced concrete retaining wall construction and arches on a 4.94 per cent grade. The work was let under three contracts, the prices being $297,792 for the two truss spans, $182,780 for the three river piers, and $264,063 for the north and south approaches, making a total cost of $744,635. The truss spans have a clearance of 70 ft. above high water. The bridge is a highway structure with a 36-ft. roadway, providing for two car tracks, and a sidewalk cantilevered from each truss. The floor construction for the truss spans consists of wood block paving on a buckle-plate floor, supported by steel stringers and floorbeams. The sidewalk floor consists of a reinforced concrete slab, supported by the fascia girder, curb girder and cross beams. The bridge has an ornamental iron railing. The shore ends of the truss spans are arranged to support ornamental stone portals. The three piers and the steel work of the two truss spans are completed, and the approaches are to be completed by July 1, 1915.

LOADS AND ALLOWABLE STRESSES.

Dead Load.—In estimating the dead load the wood block paving was taken at 18 lbs. per square foot; concrete at 144 lbs. per cubic foot; track rails and fastenings at 160 lbs. per linear foot of bridge; sidewalk slabs at 50 lbs. per square foot; sidewalk railing at 50 lbs. per linear foot each; 24-in. water mains, one under each sidewalk, 500 lbs. per linear foot each; 16-in. gas mains, one under each sidewalk, 45 lbs. per linear foot each; 12-in. gas mains, one under each sidewalk, 33 lbs. per linear foot each; and wire conduits 200 lbs. per linear foot for each sidewalk. The otal dead load for linear foot of span was aken at 14,600 lbs.

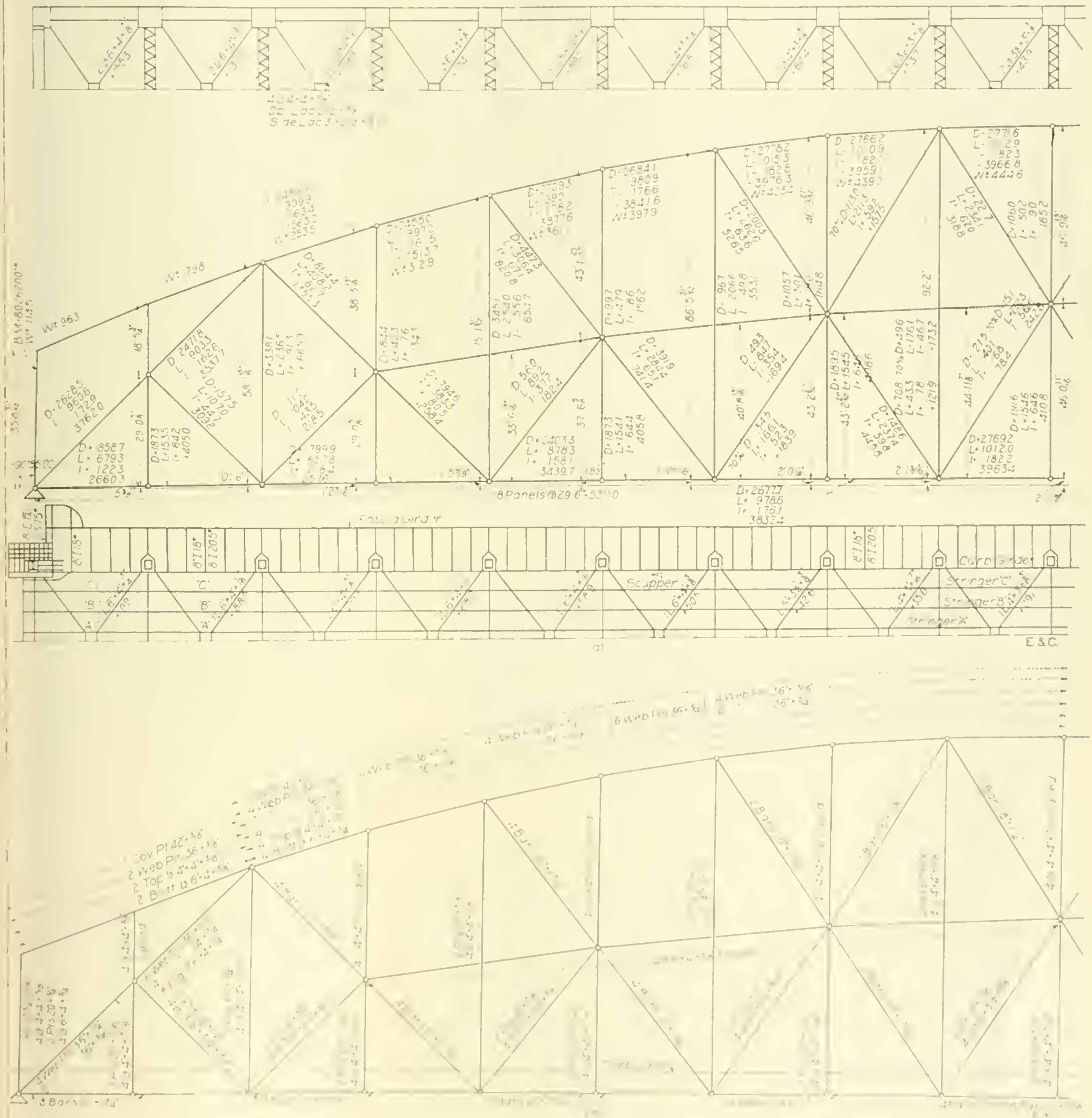


Fig. 1. General Dimensions, Stresses and Sections for 531-ft. Trusses of North Side Point Bridge, Pittsburgh, Pa.

Live Load for Floor System.—The live load er car track consisted of a continuous line f 52-ton, double-truck cars, each 45 ft. long. he wheels of each truck were spaced 5 ft. centers, trucks of car were spaced 25 ft. centers, and trucks of adjacent cars were aced 20 ft. on centers, the two tracks occupyg 18 ft. width of roadway. The live load r the roadway on each side of the tracks nsisted of a concentrated load of 20 tons on o axles spaced 10 ft. apart with a 5 ft. gage, cupying a floor space of 9 ft. wide by 18 long, and a uniform load of 100 lbs. per uare foot for all of the roadway surface t covered by the cars and the concentrated ad. The live load for the sidewalk was ken at 100 lbs. per square foot.

Live Load for Trusses—The same live load on the car tracks was used for the trusses as for the floor system, except that alternate cars were omitted. The uniform live load on that part of the roadway not covered by the car tracks was taken at 75 lbs. per square foot, and on the sidewalks it was taken at 50 lbs. per square foot. The total live load per foot of span was taken at 5,336 lbs.

Impact.—The impact stresses were computed from the formula $I = S \frac{150}{L + 300}$, where I = impact stress, S = maximum live load stress, and L = loaded length in feet to produce the maximum live load stress. Impact stresses were not added to lateral or wind forces, nor to the live loads on the sidewalk brackets.

Wind Load.—A wind pressure of 30 lbs. per square foot on the exposed surface of both trusses and on the bridge floor as seen in elevation was used, this load being treated as a moving load.

Allowable Stresses. The following allowable stresses were used: Tension, net section, 16,000 lbs. per square inch compression, $16{,}000 - 70 \frac{l}{r}$ (with a maximum of 14,000 lbs. per square inch), where l = length of member in inches and r = least radius of gyration; bending on extreme fibers of rolled shapes, 16,000 lbs. and on pins, 24,000 lbs.; shearing value for shop driven rivets, 12,000 lbs. per square inch, and for field driven rivets and turned bolts, 10,000 lbs. per square inch; shearing value for plate girder webs, gross section, 10,000 lbs. per square inch; bearing value for shop driven rivets, 24,000 lbs., and for field driven rivets and turned bolts 20,000 lbs. per square inch. Members subjected to alternate stresses were proportioned for the stress requiring the larger section. If the alternate stresses occurred in succession during the passage of one load, as in stiff counters, each stress was increased 50 per cent of the smaller stress. The connections in all cases were proportioned for the sum of the stresses. In designing counters only 70 per cent of the dead load stress was considered effective in reducing the live load stress. For stresses produced by lateral and wind forces combined with those from live and dead loads the allowable unit stress was increased 25 per cent.

TYPE OF TRUSS AND GENERAL DIMENSIONS.

The two truss spans, which are alike, are of the modified Baltimore truss type with subdivided panels (see Fig. 1). The length of span is 531 ft., center to center of end pins, this length being made up of 18 sub-panels, each 29 ft. 6 ins. long. The depth of the truss at its center is 92 ft. 10½ ins. and at the hip, 58 ft. 1 15/16 ins. The trusses are spaced 40 ft. on centers, giving a 36-ft. roadway. The width of each cantilevered sidewalk from curb to outside of fascia is 12 ft. 0 ins., and from center of truss to outside of fascia it is 10 ft. 6 ins. The total width of the bridge, out to out of steelwork, is 60 ft. 0 ins.

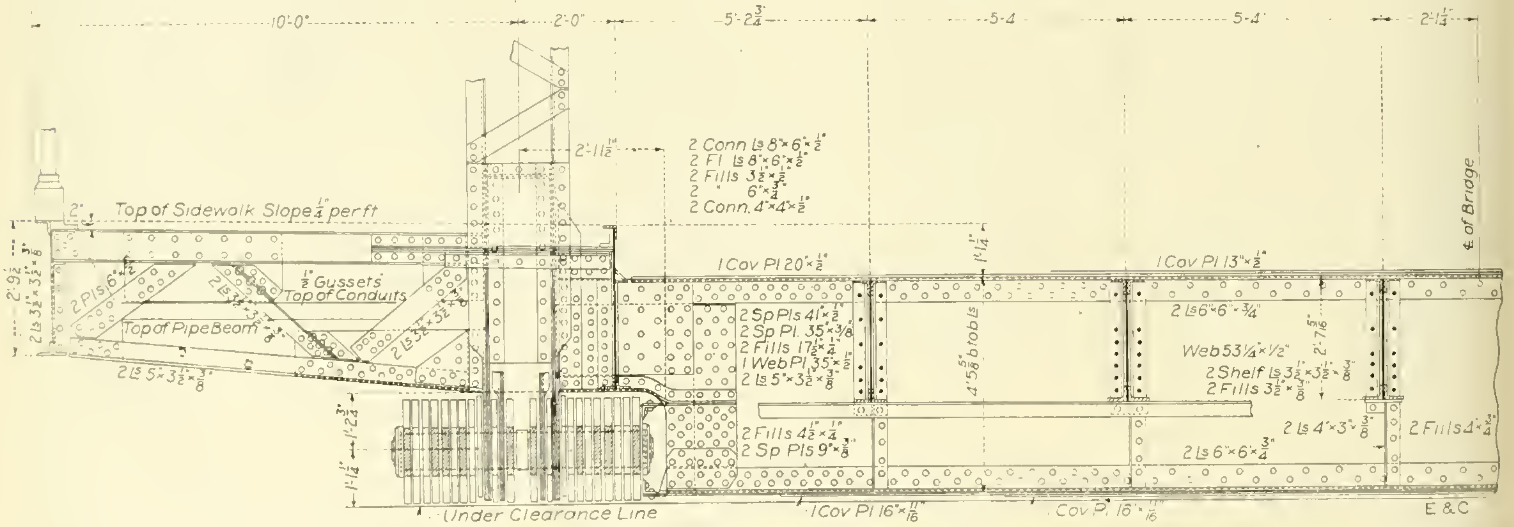


Fig. 2. Cross-Section of North Side Point Bridge, Pittsburgh, Showing Type of Floor Construction, Sizes and Dimensions.

STRESSES AND SECTIONS.

Figure 1 shows the stress sheet for the 531-ft. spans. This drawing shows the dead load, live load, impact and maximum stresses for each member of the truss, together with the section used.

By noting the material given for the upper chord section it will be seen that the section has three webs. The top flange consists of four angles and a cover plate, and the bottom flange of four angles latticed with 5x½-in. bars. The three webs are braced by diaphragms placed near the ends of each member. The intermediate posts are subdivided, and are braced by an efficient system of sway bracing and by longitudinal struts. The largest eye bars used in the lower chord have a section of 12x2¼ ins. The maximum size of pin (at L_0 and U_2) has a diameter of 12½ ins. The lower chord is cambered as shown in Fig. 1 (a), the maximum camber at the center being 2 ft. 1½ ins.

The portal consists of a heavy box-section top strut, with curved knee-braces consisting of four angles latticed together. The gusset plates extend to the knee-braces, thus affording a rigid portal. The members of the sway bracing system, consisting of four angles with double lacing, have a width equal to that of the intermediate posts. This system braces the posts at two intermediate points. The top lateral strut consists of four 4x4x⅜-in angles. The double top and bottom lacing for this strut consists of 2½x⅜-in. bars, and the single side lacing consists of 3x2½x5/16-in. angles. The members of the top lateral system are composed of two angles placed back to back, these members being riveted at their intersections. The members of the bottom lateral system consist of a single angle, and are riveted at their intersections. A heavy lower lateral system is not required as a buckle-plate floor is used.

The stringers have a depth of 30½ ins., the intermediate floorbeams a depth of 53⅝ ins., the curb girders a depth of 40½ ins., and the fascia girders a depth of 33½ ins., all dimensions being from back to back of flange angles. In computing the required areas for the flanges of the stringers and floorbeams one-eighth of the area of the web was considered effective area for the bottom flange and one-sixth for the top flange.

Figure 2 shows a transverse section of the bridge. This drawing shows the type of construction used for the sidewalk brackets and intermediate floorbeams, and gives the sizes of the members and the general dimensions. The sidewalk bracket is very heavy, as it carries gas and water mains in addition to the floor loads. One of the top cover plates of the floorbeam is 20 ins. wide and is punched to provide a connection for the ⅜-in. buckle-plate floor.

The wearing surface for the sidewalk consists of a 4½-in. concrete slab, reinforced with triangular wire fabric having a 4-in. mesh. This wire mesh is placed 1½ in. below the top of the slab at the supports and 1 in. above the bottom of the slab at the center of the span. The 8-in. 18-lb. I-beam supports are spaced 4 ft. 11 ins. on centers.

The shoes for the fixed end of the bridge are built-up members and rest on 60x1½-in. x 4-ft. 9½-in. base plates. Their height from center of pin to top of base plate is 2 ft. 6 ins.

The shoes and roller nests for the expansion end are shown in Fig. 3. This drawing also shows the connections for the sidewalk bracket and for the end floorbeam. The roller nest consists of eight segmental rollers, each 12 ins. in diameter x 6 ins. wide x 5 ft. long. A commendable feature of the roller nest is the cast-iron reservoir which surrounds the segmental rollers. This reservoir is to be filled with oil, thus insuring perfect action of the rollers. It will be noted that the bed plate rests on an I-beam grillage imbedded in the top of the pier.

COMPOSITION OF PAINTS USED FOR STEELWORK.

The following specifications covered the composition of the paint used for this bridge:

The shop paint shall be mixed in the proportion of 80 lbs. of pure dry red lead to 20 lbs. of pure French yellow ochre ground in oil, mixed with pure raw linseed oil. The weight of the paint ready for use shall not be less than 24 lbs. per gallon.

The first coat of field paint shall be mixed in the proportions of 85 lbs. of pure white lead ground in oil to 15 lbs. of pure American white zinc ground in oil, mixed with pure raw linseed oil. The weight of the paint ready for use shall not be less than 18 lbs. per gallon.

The second coat of field paint shall be mixed in the proportions of 85 lbs. of pure white lead ground in oil to 15 lbs. of pure American white lead ground in oil, with sufficient pure French yellow ochre ground in oil and pure carbon black ground in oil to give the color desired by the director, the pigment to be added to the pure raw linseed oil in such proportions that the paint when ready for use will weigh not less than 19 lbs. per gallon.

No drier shall be used in the shop paint and only in limited quantities in the field paint, all drier to be shipped to the site in separate packages and added to the paint.

All red and white lead to be 99 per cent pure, all American white zinc to be 99 per cent pure, and all carbon black to be 98 per cent pure carbon. All yellow ochre to be pure French ochre and to contain not less than 19 per cent ferric oxide and not more than 5 per cent of lime in any form. Driers used shall be pure turpentine free from resin, benzine or any mineral oils. All pigments, oils, driers and mixed paints to be subject to analysis by the City's chemist.

METHOD OF PAYMENT.

In making payments to the contractor the value of the steelwork fabricated, inspected and accepted at the contractor's shop was taken at 60 per cent of the total price per pound; the value of the fabricated steelwork delivered at the bridge site was taken at 65 per cent; and that of the steelwork erected but not riveted or painted was taken at 85 per cent of the total. It was estimated that the total weight of steel in the two truss spans, exclusive of sidewalk railing, was 4,860 tons. In determining the price per pound for making estimates it was specified that this was to be determined by dividing the lump sum price by 10,000,000. It was further specified that 80 per cent of the current monthly estimates were to be paid to the contractor after the making of these estimates; that 90 per cent was to be paid upon the completion of one span to the satisfaction of the director; and that the remaining 10 per cent was to be paid to the contractor upon the completion and final acceptance of the entire work, all payments to be made within 30 days after making such estimates and acceptances.

PERSONNEL.

The North Side Point Bridge was designed under the direction of the Department of Public Works of Pittsburgh, Mr. John G. Armstrong, director, and Mr. N. S. Sprague, superintendent. Mr. Emil Swensson was consulting engineer for the bridge, and the American Bridge Co. was the general contractor for the superstructure of the truss spans.

Bridge Work in Connection with the Elimination of the Grade Crossings of the Nickel Plate at Cleveland, Ohio.

The New York, Chicago & St. Louis R. R., commonly called the "Nickel Plate," traverses the city of Cleveland and its suburbs for a distance of 16.94 miles. Its course is intersected by 120 highways, 20 of which either did not cross at grade, when the road was constructed in 1882, or had been separated from the grade of the railroad prior to 1909. In the latter year operations were begun to eliminate the grade crossings, and this article treats of the bridge work required for the elimination of these crossings. The ornamental features of the bridges are of special interest. The data contained in the article were taken from a paper by Mr. A. J. Himes, engineer of grade elimination, in Vol. 15, No. 160, Bulletin of the American Railway Engineering Association.

GENERAL.

In Cleveland it was arranged that all street work and highway bridges should be constructed by the city through its engineering department. In East Cleveland all work was done by the railroad company. The ordinances requiring the work provided for the elimination of all grade crossings between East Ninety-third St., Cleveland, and Ivanhoe Road, East Cleveland, a distance of 4.97 miles. Of this distance the four-track roadbed covers 2.4 miles and a double-track roadbed was built the remaining distance. The work required 63,000 cu. yds. of concrete, 5,500 tons of steel for the bridges, and 7,500 lin. ft. of temporary and permanent wooden trestles. In general the concrete was a 1:3:6 mixture. For the bridge floors and for reinforced concrete work a 1:2:4 mixture was used. The steel bridge work was performed entirely by contract.

Very strong opposition to the ordinances developed among the residents of the territory where the road was to be built, which resulted in the final adoption of some very unusual bridge designs to make the structures less unsightly than the usual steel construction.

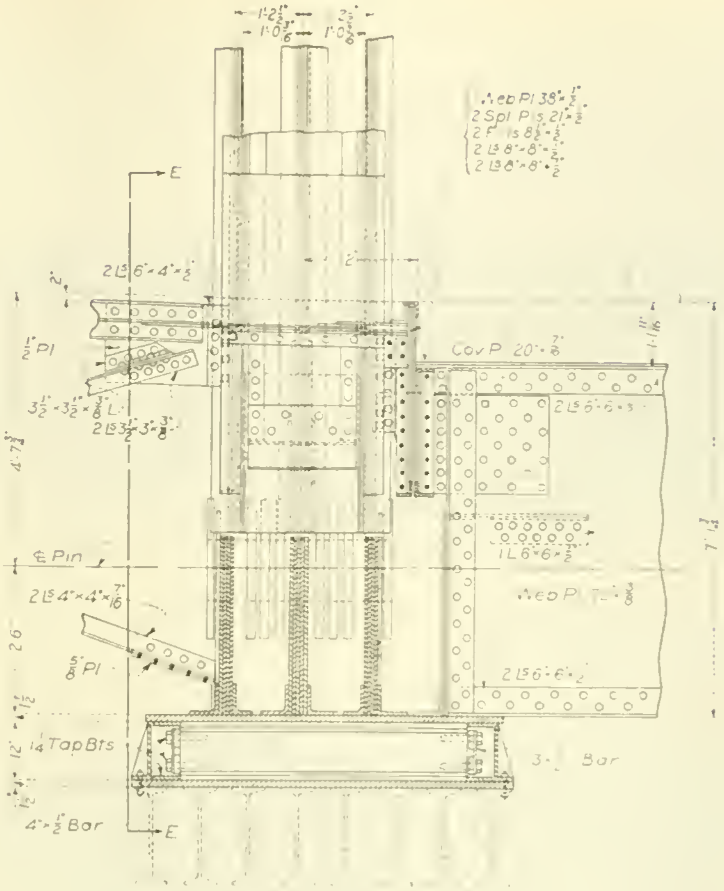
STEELWORK OF BRIDGES.

The bridges were all designed according to the New York Central Lines' specifications of 1910. The Cleveland bridges were the first to be constructed under these specifications, and a large part of the writer's work, as a member of the New York Central Lines Bridge Committee, was to secure the adoption of a joint specification in order that it might be available for these bridges.

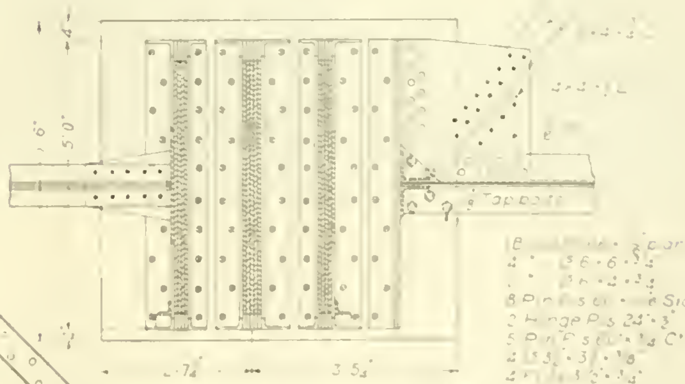
The traffic of the Short Line promised to be as heavy as any in America, while that of the Nickel Plate is lighter. The Nickel Plate is operated wholly independently of the New York Central Lines, but its relations with the latter are so close that to build any portion of the four-track bridges for less than the maximum requirements seemed very short-sighted. Joint specifications seemed the easiest means of securing authority for proper bridges.

The live load in these specifications is Cooper's E-60, with an alternate load of [illegible] lbs. equally distributed on two axles spaced 7 ft. center to center.

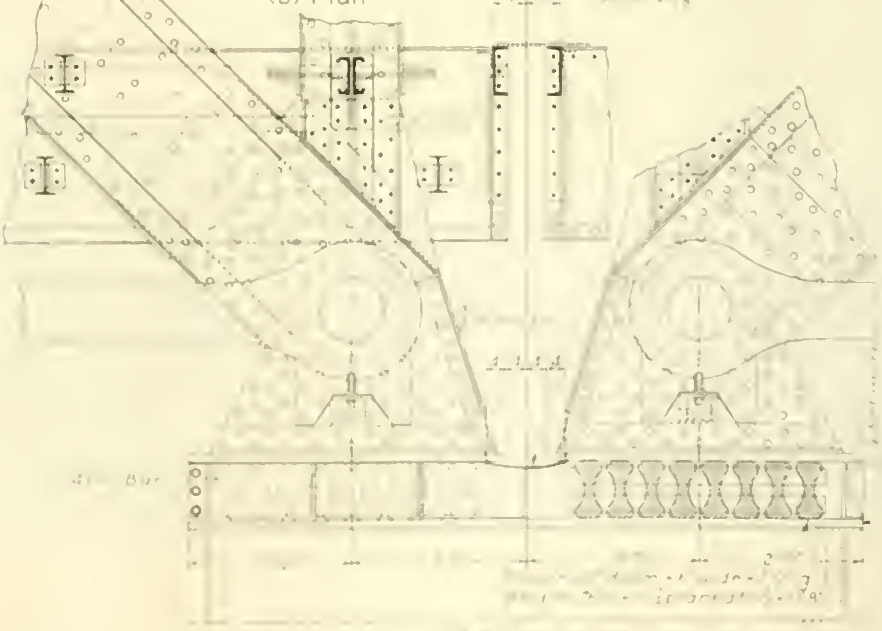
The unit stresses are: Tension, [illegible] lbs. per square inch; compression, [illegible] but not to exceed 15,000 lbs. per square inch; and the impact $I = S\left(\frac{300}{L + 300}\right)$. The use of unit stresses somewhat higher than common is in recognition of the apparent impossibility of any material increase of live load without a general reconstruction of all railway structures.



(a) Section at Expansion Ends



(b) Plan



(c) Section E E

E&C

Fig. 3. Cross-Section, Plan and Elevation of Expansion End of North Side Point Bridge, Pittsburgh.

The bridge floors are of I-beams, either encased in concrete or bearing a concrete slab above the beams.

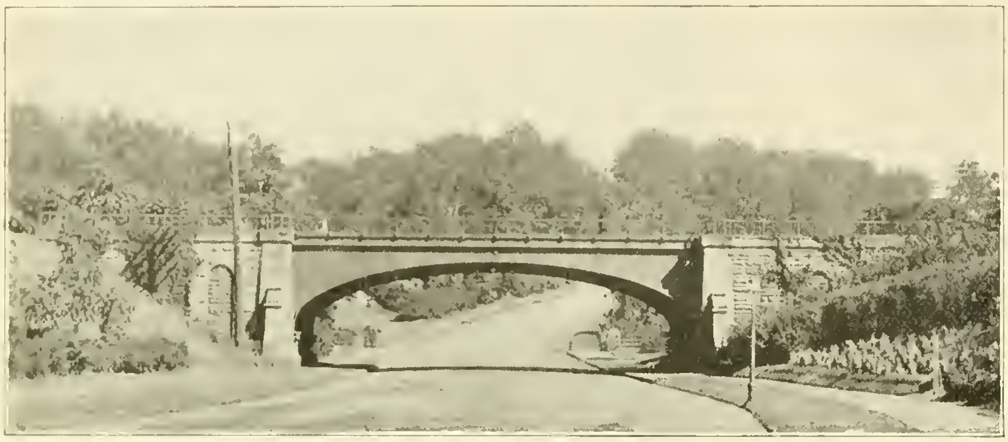


Fig. 1. Old East Boulevard Three-Hinged Steel Arch, Cleveland, O.

Fifteen of the bridges were plate girders; seven of them were three-hinged arches.

The former East Boulevard Bridge of the Nickel Plate (Fig. 1) was considered a very handsome structure, and the people of East Cleveland, in the hope of beautifying their city, demanded the construction of similar arch bridges. To this the company objected strongly, but it was finally arranged to build such bridges on a few particular streets. There was no material difference in weight between the plate girder and arch bridges. There was a considerable difference, however, in the volume of masonry in the foundations. The rock surface was about 12 to 16 ft. below the surface of the streets, and the foundations of the arch bridges were designed to carry the arch thrust to the rock. This increased the volume of concrete materially above what was needed for the plate girder bridges.

No lateral bracing was used in any of the bridges, it being left for the floor slabs to furnish lateral rigidity. Before the construction of the floor slabs temporary wooden stringers were laid on the I-beams to support the track to grade, and the trains caused considerable motion in the arch bridges and the girders having curb supports. In all such cases temporary wooden bracing was used to check the motion.

At Euclid Ave. a combination of long span, sharp skew, curve and heavy loading made it necessary to use curb supports. Curb supports were likewise used at the Mayfield Road.

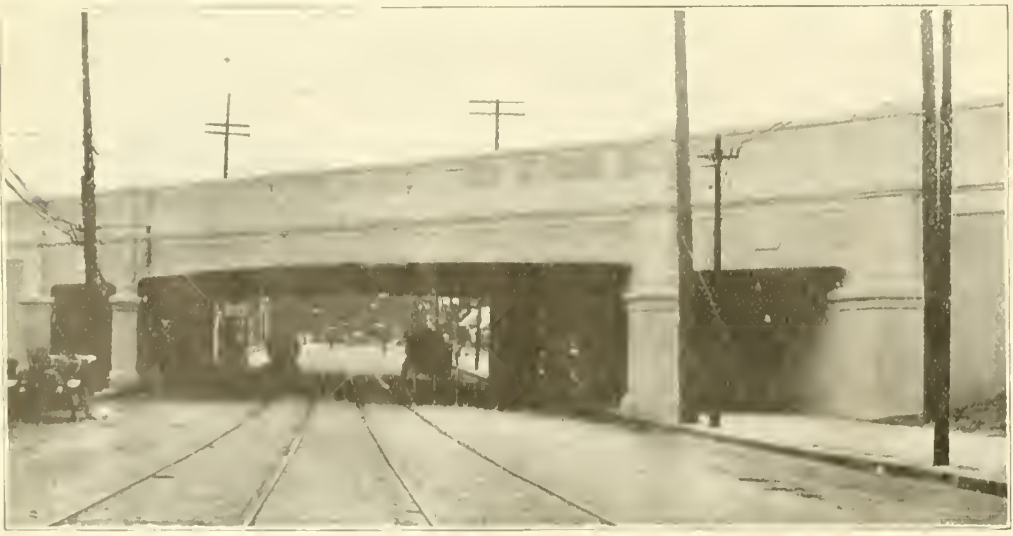


Fig. 3. Euclid Ave. Bridge—Plate Girders with Concrete Fascias—Steel Columns Encased in Concrete.

The old bridges at Cedar Ave. and East Boulevard were dismantled, and all new bridges were erected with derrick cars. All bridge work was done by contract.

The steelwork of the highway bridges consisted mainly of lattice columns and longitudinal beams, the whole being encased in concrete. The tonnage of the railroad bridges was 4,870, and that of the highway bridges 630—a total of 5,500.

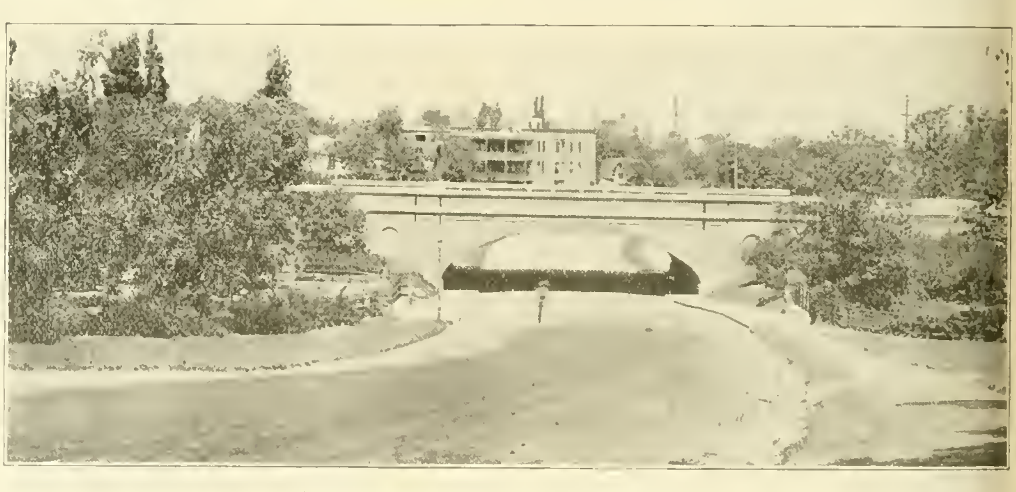


Fig. 2. New East Boulevard Bridge, Cleveland— Combination of Arch and Plate Girders with False Parapet and Soffit.

DESCRIPTION OF EAST BOULEVARD AND EUCLID AVE. BRIDGES.

East Boulevard Bridge.—The original East Boulevard Bridge was a three-hinged plate girder arch with ornamental stone abutments. It was built for two tracks and had a clear span of 57 ft. 4 ins. The floor consisted of I-beams with a deck plate to which the rails were fastened directly by clips and bolts.

The revised grade of the railroad contemplated a lowering of the Nickel Plate track over the boulevard of 6.39 ft. The grades of the two roads separate just east of Cedar Ave., the Nickel Plate descending sharply to the westward on a 0.5 per cent grade, and the Short Line ascending westward on a 0.3 per cent grade. The effect of this divergence at the boulevard is to produce a difference in the elevation of grade lines of 3.72 ft.

The city ordinance provided for a plate girder bridge with concrete abutments, but when the time came to detail the work the park department entered a protest. It desired that the bridge be rebuilt on lines exactly similar to those of the former bridge. It was impossible to do this, as was reluctantly admitted after a careful study. A stone arch was then proposed, with abutments similar to those of the old bridge. This was impracticable because of insufficient space from top of rail to soffit on the Nickel Plate side. The negotiations continued a whole year without results. It was finally proposed by Mr. Hoffman, chief engineer of the Board of Public Service, that a composite bridge be built; that a reinforced concrete arch be built for the Short Line tracks where there was plenty of room, and that plate girders be used for the Nickel Plate tracks. The present bridge (see Fig. 2) is the outcome of that suggestion. It was designed to meet the requirement that it have something of the appearance of the old structure, except that the arch should be concrete instead of steel. To carry out the idea as well as possible the writer offered to surround the plate girder with a parapet and false soffit, so as to present the appearance of a simple arch structure and to face it with a matrix of red granite. Later it was decided to color the mortar with iron oxide—not a fortunate proceeding, for the color is much inferior to that of the granite. The granite is now exposed in the rough panels and along the moldings, pilasters and bases where it is bush-hammered. The smooth surfaces are weathering gradually and after a time the difference of color will be less pronounced.

The design of the structure is of interest. The arch is solid and perfect in condition, except for a fine vertical crack in the parapet over the haunches. This is a shrinkage crack and it occurred soon after construction. Its position could have been predetermined by a joint. The whole arch was formed continuously without any intermission night or day, and has thus far been wholly impervious to water, although no waterproofing was used.

Because of the deflection and vibration of the plate girder, a longitudinal joint was constructed along the face of the arch and adjacent to the plate girder to avoid cracking. The joint was filled with oakum and an asphalt mixture, but it will not stay in place. The motion of the steel span causes it to work upward out of the joint.

The false soffit under the plate girder is divided into three sections. The center is a part of the concrete floor slab. There is a transverse joint 10 ft. 8 ins. each side of the center. From the joint to the springing line the soffit is a sheet of mortar 3 ins. thick, plastered on woven wire. The wire is supported by a framework of light channels. Provision is made for motion at the transverse joints. The concrete parapet or face of the false arch is separated from the concrete floor slab by a vertical joint. This joint has thus far prevented any damage to the concrete due the the vibrations from passing trains.

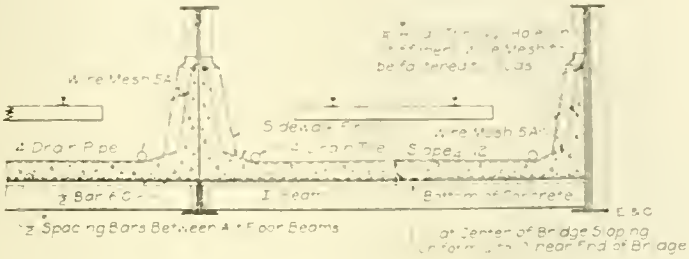


Fig. 4. Section of Typical Bridge Floor—Concrete Slab and I-Beams.

Euclid Ave. Bridge.—At Euclid Ave. the tracks cross the street on a 4° curve, the tangent making an angle of 37° 29′ with the center line of the street. The street was depressed 3 ft. 6 ins., and the railroad grade was elevated 14 ft. 4 ins. This street was made a controlling point in the grade line, the prime object being to avoid as far as possible any obstruction to the view along the avenue. In this respect the result is quite satisfactory as may be seen in Fig. 3.

Preliminary studies indicated that a plate girder bridge with curb supports was the only feasible design. Sketches were made for trusses spanning the whole street, but they were objected to as unsightly. The clear span measured on the skew is 140 ft. 3 ins. Because of the curve it was necessary to space the girders 19 ft. center to center. The heavy live load and the enormous dead load, together with the lateral clearance needed because of the curve, made the use of curb supports imperative. Solid shale was found 27 ft. below the old street surface. It was overlaid with water-bearing sand, the depth of water varying with the season—generally being about 6 ft. A concrete pier was constructed under each of the girders on the curb line, ten piers in all being provided. The inside piers are 6½x6 ft. in plan, and carry an estimated maximum load of 17 tons per square foot. The sidewalk spans are so short that the abutments are little more than retaining walls. Their load is small, but to insure against any settlement which might damage the concrete superstructure they are founded on piles.

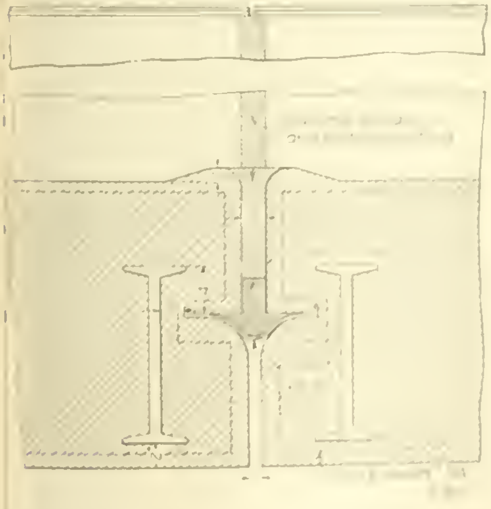


Fig. 5. Detail of Joint in Floor Slab for Arch Bridge.

The thickness of the floor, 3 ft. 6 ins. from top of rail to the underside, was agreed to by the city officials a long time before the passage of the ordinance, even before the live load and unit stresses were determined, and before the type of bridge had been selected. When the design of the structure came up for consideration it was soon found that the depth of floor allowed was very scant. A minimum thickness of ballast of 8 ins. had been agreed to as necessary to prevent noise and avoid damage to the concrete. The summit of the grade had been located on the bridge so it was not desirable to raise the grade. These conditions are stated in detail because, even though investigations had indicated that the waterproofing of the floor slabs was in ordinary cases unnecessary, there was no fall to take water away from the bridge in this case, and cracks in the floor were bound to occur over the curb supports. Had there been sufficient depth of floor available to waterproof the slab and give it a protective covering, such a course might have been followed. Under existing conditions the leakage at the curb line is considerable.

Fig. 6. Cedar Ave. Bridge—Plate Girders Masked by Concrete.

The bridge was constructed without at any time closing the street, only one street car track and one side of the roadway being abandoned at a time while depressing the roadway. The inside girders weighed 79 tons, and measured 91 ft. 3¾ ins. over all. They were placed with a derrick car from above.

The form of the bridge in plan is a rhombus, the diagonals being respectively 81½ ft. and 215½ ft. Both ends of the structure were left free on the abutments. The tops of the abutments were finished smooth and were well painted with heavy asphalt paint. The floor slab was then extended over the abutments. Some trouble was expected from temperature changes, and trouble did later develop. There has been a little motion on the top of each abutment, just enough to crack the mortar in the angle between the floor and the abutment face. The greatest motion has occurred at the extreme apices of the rhombus. At the northerly apex the motion has resulted thus far in a few cracks that are hardly noticeable. At the southerly apex, in February, 1912, a crack appeared in the face of the pilaster extending from the upper right-hand corner to the lower left-hand corner, although two expansion joints had been constructed in the abutment. One of the joints did not appear to work, and it was naturally assumed that the crack was occasioned by the tendency of the wing walls to part from the head wall in the angle, which is a common occurrence in abutment masonry. To remedy this the wing wall was cut loose from the head wall during the following summer by drilling entirely through the abutment, and the crack in the pilaster was filled to a depth of 2 or 3 ins. In the winter of 1912-13 the crack again opened, and further repairs were required. The portion of the pilaster below the bridge seat and between the crack and the joint near the end of the abutment has been removed and rebuilt, using reinforcing bars and dowels in the old concrete. At the bridge seat two grillages made of rails have been placed, one bearing upon the other, the rail heads being in contact with each other. In this way we have endeavored to provide a sliding surface of less resistance, it being thought that the pull of the bridge in cold weather was the cause of the crack. The portion of concrete which was removed from the pilaster showed a projection from the face of about 1/16 in. in extreme cold weather.

Another crack, and one which should have been avoided, occurred in the outside columns. The bridge proper is carried on steel columns encased in concrete. These rest on concrete piers, carried to the rock foundation. The concrete fascias which conceal the bridge rest on the outside columns, but the columns were sufficiently elongated in cross-section with concrete to carry the load. In each of the four outside columns the concrete has parted from the steel, although the crack is, as yet, barely visible. It is now evident that steel reinforcement should have bound the concrete under the fascia to the steel column under the girder, even though the composite column does have a solid rock foundation.

BRIDGE FLOORS.

It was required that the railroad bridge floors be relatively noiseless and waterproof. In former years every effort has been put forth to build shallow floors so as to minimize the change of grade. Such floors have always permitted the muddy water to seep through upon people passing below and have operated as drums in accentuating every sound from the passing trains. To overcome these defects the floors were made of I-beams and concrete slabs, upon which tracks were laid and ballasted as upon the ground. Such a design requires a greater depth of floor, which means a greater change of grade, and more steel to carry the added weight of concrete. The bridge is therefore more expensive. But in cities where the noise is troublesome the ballasted floor is a great improvement. Trains passing over such floors are noticed but little more than when passing over the solid ground.

A concrete floor slab can also be made reasonably water-tight. Both observation and experience indicated that water-tightness could be secured either by concrete alone or in combination with waterproofing. The requisite seemed to be that the material and workmanship should be the very best. If poor waterproofing were placed over poor concrete the structure would leak. If the concrete were good it would hold water, either with or without the waterproofing.

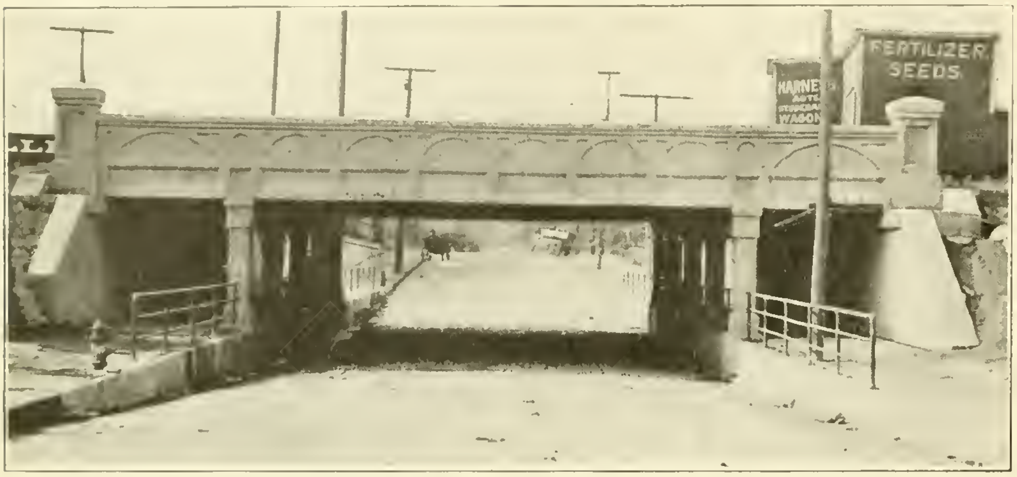


Fig. 7. Mayfield Road Bridge—Plate Girders Masked by Concrete.

In order to confirm these opinions before construction an investigation of concrete practice on other roads and in building work was undertaken by Mr. G. H. Tinker, bridge engineer, New York, Chicago & St. Louis R. R. Later as chairman of the Committee on Masonry of the American Railway Engineering Association he had exceptional opportunities for continuing the study. The result of this work was in harmony with the above-stated views, and the bridge floors were accordingly designed without waterproofing. A paper by Mr. Tinker, setting forth briefly the information he has accumulated, appeared in Vol. 5, No. 3, of the Journal of the Cleveland Engineering Society. The following extract is quoted therefrom:

> In the bridges recently built in Cleveland by the Nickel Plate no foreign waterproofing substance has been used. An attempt has been made to construct a concrete slab which would be in itself as nearly waterproof as is practicable or desirable to make. This has proved satisfactory. When the Cedar Ave. bridge floor was built, the ends of the bridge were dammed up, and the trough so formed was filled with water and allowed to stand for several days. No water whatever came through at any point of the slab. A little water ran through the dam and down over the back wall and seeped through the joint between the bridge seat and floor slab. At the center bent there is a drainage system provided to carry what water might percolate through at that point down to the gutters. Through some slight defect in the formation of this drainage some water seeped through there and dampened the concrete, but at no point of the bridge did any water drip.

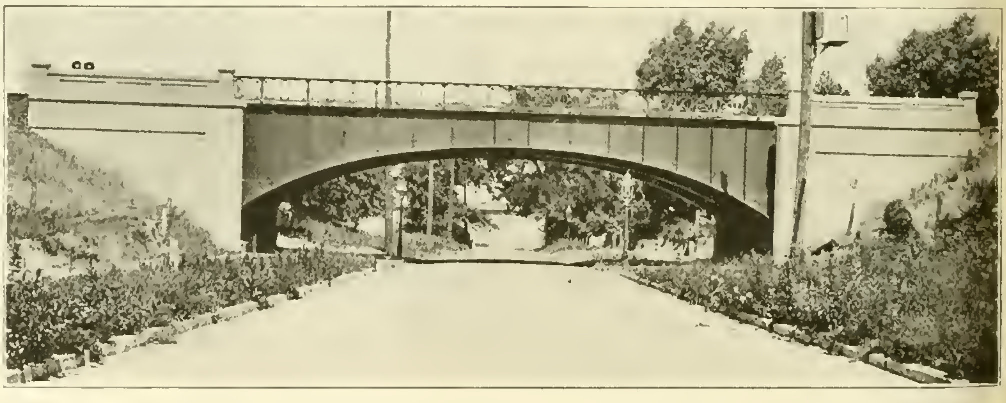


Fig. 8. Three-Hinged Steel Arch at Eddy Road, Cleveland.

Special efforts were made to avoid the entrance of water between the steel and concrete and at points of contraflexure and where cracks might develop from temperature changes. Bevel flashings of steel were riveted to the girder webs and malleable cast flashings were fitted around the stiffeners to cover and seal the edge of the concrete (see Fig. 4). This design was very successful. At points of contraflexure over curb supports and at Cedar Ave. over the center columns it was realized that cracks would develop, and an attempt was made to forestall their appearance by the construction of joints. The joints were carefully provided with gutters and drainage pipes, and it was hoped that no trouble would be experienced with the water. The cracks were successfully forestalled, but the drainage was unsuccessful. The channels soon became clogged with cinders, and the details of the steelwork in the cross-girders did not leave room for a sufficient body of concrete, and in some instances the concrete proved imperfect. Therefore, while the slabs proved generally tight, there has been some leakage at points of contraflexure.

Much reliance has been placed on the use of direct labor and carefully selected foremen, but there came a great rush of work at a critical time and the floors suffered. In East Cleveland at a later date it became necessary to build waterproof joints at the hinges of the arch bridges. This was successfully accomplished in the manner shown in Fig. 5.

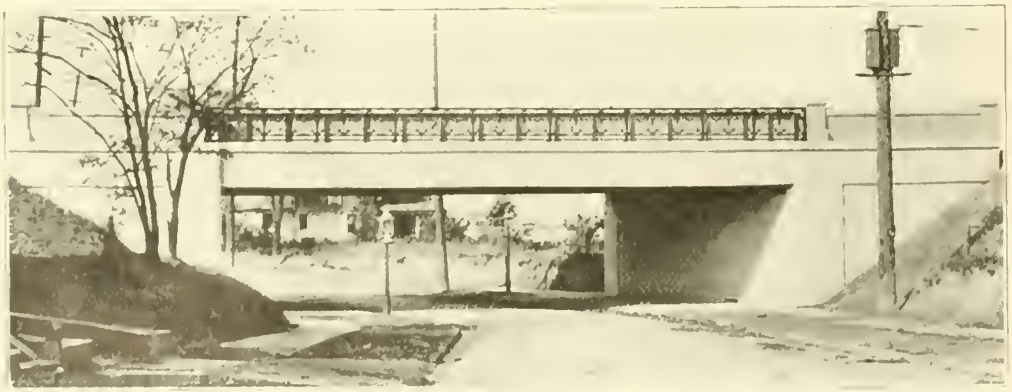


Fig. 9. Shaw Ave. Bridge, East Cleveland—Flat Beam Fascia with Ornamental Iron Railing.

The concrete in the floor slabs cost about $12 per cubic yard in place.

Now that the bridges are completed and have been two or more winters in service, the conclusions are as follows:

(1) Concrete can be made water-tight, under low heads, for all practical purposes.

(2) The mixing, placing and ingredients of concrete are subject to such a great number and variety of defects that only the keenest attention will secure an impervious structure.

(3) Contraflexture, temperature changes and settlements will produce cracks.

(4) It is best to forestall cracks with predetermined joints.

(5) Joints can be sealed against water if well designed.

The highway bridges were paved with brick. The gutters have a good fall, and the water runs off quickly. On the under side of the bridge floors the concrete is protected from locomotive blasts by cast-iron plates ½ in. thick and 36 ins. wide. They weigh 71 lbs. per linear foot, and cost $5.23 per foot in place.

ORNAMENTATION OF BRIDGES.

Both cities insisted that the bridges be of an ornamental character. In Cleveland that idea seemed to mean that the structures must be masked with concrete. Steel was held to be unsightly, but concrete was in high favor The bridges over East Boulevard and Cedar Ave. were surrounded by park land of considerable beauty. In East Cleveland the municipal artist pinned his faith to a steel arch.

A study of the bridges shown in Figs. 1, 2, 3, 6, 7, 8 and 9 furnishes sufficient evidence that bridges can be constructed which are artistic and which do not detract from the beauty of the surroundings. The bridges at East Boulevard and Euclid Ave. are shown in Figs. 2 and 3 respectively, both of which have already been described somewhat in detail. The Cedar Ave. Bridge (see Fig. 6) was first designed and planned as a type. It consists of a set of plate girders masked in concrete. The aim in its design was to secure a pleasing effect from general lines and without fineness of detail. The restrictions of space for the roadway prevented that freedom of treatment which is necessary to secure the best results.

The writer has always felt a strong repugnance to the use of stone as a beam. Such a beam is not self-supporting and can never be more than a symbol of deception. Accordingly he tried to relieve the "curse" by giving the fascia the form of an arch, but, while the curve of the soffit is somewhat pleasing, the required clearance of the roadway prevented a rise that would afford much resemblance to an arch.

The reddish color of the East Boulevard Bridge, designed about a year later, resulted from some criticism of the glaring whiteness of concrete.

The elevation of the concrete fascia of the Mayfield Road Bridge (see Fig. 7) was designed in the office of Robert Hoffman, chief engineer, Department of Public Service. In this case the concrete fascia is built directly upon the outer girder, and it was desired to cover the stiffener angles and to secure a pleasing effect without the use of too large a mass of concrete.

In East Cleveland the artists were not so active. The attitude was rather a stubborn opposition to everything proposed by the company. Steel arches were demanded because of the one formerly used at East Boulevard, and a type of iron railing was designed and adopted by the council. The company was finally able to avoid all but seven of the arches. It had little interest in the railings.

Figure 8 shows a view of the Eddy Road Bridge, which consists of a three-hinged steel arch with an ornamental iron railing.

A band of concrete—a concrete beam—was used to conceal the plate girder spans of the Shaw Ave. Bridge shown in Fig. 9. This bridge also has an ornamental iron railing.

Since the bridges have been completed the consensus of opinion locally is in favor of the plate girders, and it is admitted that the lines of the railings are too fine.

Railways

Principles and Methods for the Valuation of Railroad Property and Other Public Utilities.

(Continued from Page 316.)

OVERHEAD CHARGES.

When the items contained in the inventory are multiplied by the corresponding unit prices and the products are added, the result is only a part of the cost of creating the physical property, for the reason that the unit prices used are generally based upon the direct cost of similar works, and do not include the indirect or incidental expenses. For instance, in fixing a price for a building, the unit price under most circumstances would be the amount for which a contractor would undertake to build and complete the building, or it might be the estimated cost of supplying the labor and materials for the building, with a due allowance for the profit and expenses of the contractor and sub-contractors. In order to obtain the full cost of a building, it would be necessary to include the fees of architects, the amount paid inspectors, the cost of grading the grounds, the amounts expended for interest, taxes, insurance, and legal expenses, and a fair sum for the value of the time devoted by the owner to the planning and construction of the building.

If the person making the valuation had information as to the cost of a similar building which included all these items there would be little, if anything, to be added to obtain the reproduction cost of the building, but if, as is usually the case, the unit cost does not include such items, material additions must be made in order to determine the true reproduction cost.

The amount of the overhead charges applicable to any property should be determined on the basis of the prices prevailing at or near the time of the valuation, but under the conditions existing when the plant was created. The accurate determination of the amount of the overhead charges presents such difficulty that reasonable actual costs should be used as a basis in the case of fairly new properties where such costs can be determined.

The allowance for overhead charges has generally been substantially underestimated by commissions and the courts, as well as by engineers of limited practical experience in construction work. The latter are prone to assume that unit prices bid by contractors or determined during the progress of the work by the inspector are fair measures of its final cost to the owner, but such is rarely the case.

The character and amount of overhead charges are discussed in the following sections:

PRELIMINARY EXPENSES.

Before a public service property can be brought into existence or even its construction begun, a considerable amount of money is necessarily expended for planning, investigating, organizing and financing [illegible] this expense, together with the [illegible] of those promoting and organizing the work, should be included in the [illegible]. The promotion of projects, public [illegible] has so often been accomplished [illegible] fraudulent methods that [illegible] carries with it unpleasant [illegible]. It [illegible] be recognized, however, that [illegible] man or body of men to pr[illegible] projects they would not exist except under public ownership, and that the services of the promoter are therefore valuable.

The preliminary stage of important developments is often long continued and expensive. For instance, in the case of a large hydro-electric development it is necessary, first, that someone should discover a situation favorable for the development of power and near enough to a market to warrant the consideration of the project. With such a project several steps are usually taken.

First, there is the preliminary study, which must be carried to the extent necessary to warrant expenditures for a further investigation and for obtaining a charter or franchise. Next there is usually a further investigation, with engineering and legal assistance, options are obtained upon the land required for reservoirs and other purposes, reports are prepared, and in general the project is put in shape for presentation to those who may be willing to finance such an undertaking.

At this stage of the proceedings experts are frequently employed to make investigations of the various features of the project, and if their reports are sufficiently favorable, the organization of the corporation is perfected, the bonds are offered for sale, all necessary permits from public authorities are obtained, the acquisition of land and rights is begun, the final surveys and investigations are made, and the preparation of plans and specifications for the construction of the work is carried forward.

If the reports of the experts are unfavorable, the project is usually abandoned, with a considerable loss to those promoting the project. Promotion is a hazardous undertaking, and this feature should be considered in determining the value of the services of the promoter.

The issuing and marketing of bonds involves the payment of a commission to brokers as well as various other expenses, all of which should be included in the valuation, but the discount on bonds due to fixing the interest rate so low that the bonds will not sell at par, should not be so included.

The amount of the preliminary expenses for different projects varies both with the character of the works to be built and the degree of care and skill used in the preparatory work.

The preliminary expenses in connection with ordinary additions and extensions are generally much less than those incurred in connection with the original plant, but they still exist, as for instance, for investigations and designs, for issuing and marketing bonds, and for many other things.

The principal incidental expenses during construction may be classified under the heads Engineering, General Expenses, Contingencies, and Interest and Taxes.

Engineering. Under the head Engineering is usually included not only the engineering expenses but those of other technical employees and of inspectors. It is difficult to draw the line between the preliminary engineering and that during construction, and it is probably best in most cases to include the part of the preliminary engineering expense connected with the preparation of the final design of the work with those incurred during their construction.

The percentage of the cost of the work represented by engineering differs with the character of the works and with the amount of care and skill exercised in their design and construction. On railroads it is commonly estimated that the engineering cost will amount to 5 per cent of the physical valuation of the property, exclusive of overhead charges. Statistics of the cost of engineering are available in connection with several municipal and other works, as follows:

Metropolitan Water Works, Massachusetts. This property, costing to the end of 1912, $42,036,000, is to the extent of $15,300,000 made up of old works purchased from the City of Boston and others, on which the engineering charge is not known, leaving $26,736,000, of which the engineering charge was $2,077,000, equal to 7.77 per cent. Based on the total cost, exclusive of engineering, the percentage is 8.42. These amounts include both the preliminary engineering and that during construction.

Boston Subways. Table I has been compiled from official reports.

TABLE I. BOSTON TRANSIT COMMISSION. (1895-1912.)

	Total cost.	Petg. of total cost.	Petg. of construction cost.
Subway:			
Engineering	$ 407,475.48	9.88	11.34
Gen. expense (inc. commission) ..	131,681.87	3.19	3.68
Construction	3,586,002.33	86.93	100.00
Total	$4,125,159.68	100.00	.....
East Boston Tunnel:			
Engineering	$ 191,466.57	5.90	6.62
Gen. expense	161,134.78	4.96	5.57
Construction	2,894,595.01	89.14	100.00
Total	$3,247,196.36	100.00	.....
Boston Tunnel and Subway:			
Engineering	$ 417,866.25	5.05	5.48
Gen. expense	226,141.57	2.73	3.58
Construction ..	7,623,206.56	92.22	100.00
Total .	$8,267,514.38	100.00	.
Cambridge Connection			
Engineering .	$ 96,575.16	7.10	8.05
Gen. expense .	62,355.[illegible]	[illegible]	5.20
Construction	1,199,904.39	88.31	100.00
Total	$1,358,8[illegible].65	100.00	

New York Water Works, now in process of construction. The disbursements upon this work to the end of September, 1913, amounted to $104,178,000. The engineering expense directly attributable to the work under construction amounted to $10,050,000, equal to 9.78 per cent of the total disbursements for this work. In addition, there were engineering expenses relating to investigations of other drainage areas, which are only indirectly attributable to the work under construction, amounting to $394,000 or 0.38 per cent of the total disbursements, making the total for engineering 10.16 per cent. It is to be noted, however, that there is included in this case under the head "engineering" the cost of unusually extensive surveys and investigations which were not included as an engineering charge upon the Metropolitan water works. There is not included, however, the cost of expensive investigations by sinking shafts at the Hudson River which although originally charged to the engineering account, has been deducted [illegible] the shafts afterward became a part of the final construction.

Charles River Basin Commission, Boston.

Dam, lock, embankments, marginal conduits, etc., completed 1909. Table II.

TABLE II.

	Total cost.	Pctg. of total cost.	Pctg. of construction cost.
Administration	$ 94,011.97	2.6	3.2
Engineering	446,096.03	12.4	15.5
Construction:			
Preliminary	10,783.86	0.3	....
Contracts	2,629,671.95	73.1	....
Additional	237,433.22	6.6	....
Total const'ct'n.	2,877,889.03	80.0	100.0
Real estate	179,730.77	5.0	6.2
	$3,597,727.80	100.00	....

Pennsylvania Railroad Tunnels. The cost of engineering for the East River Division of the Pennsylvania Railroad tunnels amounted to 5.8 per cent of the total cost of the work, including excavation, retaining walls around the station area in New York, the tunnels eastward under the streets and East River to the surface in Long Island, terminal yard in Long Island, and engineering. This is equivalent to a 6.1 per cent of the cost, exclusive of engineering.

Kennebec Water District, Maine. New gravity water supply, with auxiliary steam pumping plant, 1906. Table III.

TABLE III.

	Total cost.	Pctg. of total cost.	Pctg. of construction cost.
Administration	$ 6,161.20	2.30	2.53
Engineering	17,872.35	6.69	7.35
Construction:			
Rights of way	962.97	0.36	100.00
Pumping station	20,833.39	7.80	
Steam plant	6,275.59	2.34	
China Lake Pipe Line	215,302.93	80.51	
Total	$267,408.13	100.00	.....

Louisville, Ky., Sewerage Works, 1906 to 1912. This work consisted of intercepting sewers, trunk sewers, and a small proportion of lateral sewers, Table IV. This total is exclusive of the cost of preliminary engineering, which involved an expense of approximately $57,000, equivalent to 1.7 per cent of the construction cost ($3,289,330.89) shown above.

TABLE IV.

	Total cost.	Pctg. of total expenditures.	Pctg. of payments to contractors.
Administration	$ 78,025.03	2.09	2.37
Engineering	336,544.87	9.00	10.23
Rights of way	12,319.36	0.33	0.37
Castings and other metal work	15,161.29	0.41	0.47
Damage suits (exc. of rights of way)	8,307.52	0.22	0.25
Amounts of payments to contr's	3,289,330.89	87.95	100.00
	$3,739,988.96	100.00	.....

Watertown.—Mechanical filters, built in 1903; cost, $97,065; engineering cost, 5.9 per cent.

Ogdensburg.—Sand filters built in 1910; cost of work, $167,694; engineering charges, 7 per cent, excluding cost of preliminary report, which amounted to $500.

Hudson River State Hospital.—Sand filter plant built in 1904; cost, $36,000; total cost of engineering, 10½ per cent.

Peekskill.—Sand filters built in 1908; total cost, $63,304; cost of engineering, 7.1 per cent.

Yonkers.—Open sand filters, built 1903; total cost, $50,165; total cost of engineering, 7.3 per cent.

Yonkers. Covered sand filters, built in 1907; total cost, $106,708; cost of engineering, 8.7 per cent.

Ithaca, N. Y.—Filters built in 1903 on a percentage basis under rush conditions; cost, $192,114; engineering cost, 7 per cent.

Springfield, Mass.—Water works built in 1910; construction of additional supply from Little River, including diversion works, reservoir, filters, and pipe lines; cost, $1,465,393; cost of engineering, 10 per cent. The basis of computation does not include the sum of $268,000 paid for land, legal, and other expenses.

Springfield, Mass.—Ludlow filters; built in 1906 at a cost of $43,306, to meet an emergency, and requiring very rigid inspection to secure proper grade of sand and proper sanitary conditions during construction; cost of engineering, including board, livery, cots, bedding, and provisions for inspectors on work, 17 per cent.

It is obvious that the cost of engineering varies with the character of the work. For instance, the construction of an important dam or aqueduct, built in place and requiring skill in designing, and a careful inspection of every part of the work as it is built, requires a larger expenditure for engineering than a large cast-iron pipe line where the cost of laying the pipes in a trench is but a small percentage of the total cost of the line, and the work progresses so rapidly that the inspection cost is small in proportion to the total cost.

The cost of engineering varies not only with the class of work but with the character of the design and execution. For instance, works may be built with little inspection, from crude designs prepared by unskilled engineers, with the result that the cost of works may be large although the percentage paid for engineering may be small. Work skillfully designed and efficiently constructed necessarily involve a larger cost for engineering, which should be recognized in any valuation when the works give evidence of such skill and efficiency.

The cost of engineering for additions and extensions of a property may be as large, or even a larger percentage of the total cost than for the original plant, but under a continuous system of regulation engineering for additions and extensions, if charged to current expenses, should not be included in the valuation.

It is sometimes suggested that no engineering charge should be made in connection with the acquisition of real estate, and there may be some cases where such a view would be substantially correct. It is frequently the case, however, especially where strips of land are to be acquired, that the engineering cost is as great as for other portions of the work.

The record of the cost of the Wachusett Reservoir, where the engineers had little to do with the acquisition of land other than to prepare the plans of the lands and to make appraisals of the mill property and water rights, amounted to about 3 per cent of the total cost of the property acquired. The percentage would have been considerably larger if nearly all of the property had not been acquired by purchase and very little by condemnation proceedings.

The records of the New York Water Works show that the engineering connected with the acquisition of land to the end of September, 1913, amounted to 1.87 per cent of the total expenditures on land account and to about 3 per cent of the awards to the owners of the property.

General Expenses.—Under the head of General Expenses are included all administrative, legal and other general expenses during the period of construction. Such expenses include general office rent, the salaries of the officers of the corporation and of the secretary, treasurer, legal advisers, clerks and others. Statistics showing the amount of such expenses in the case of corporations are not available to the committee. The statistics of municipal public service properties throw some light upon the subject, although generally defective in that the offices are frequently in public buildings where no rent is charged and the financial and legal duties are performed to a large extent by those who receive salaries paid out of general funds.

On the Massachusetts Metropolitan Water Works, the administrative and expert services, with a small portion of the legal services, amounted to 1.43 per cent of the cost of the work, an unusually low figure, as there was very little litigation in connection with the work.

On the New York Water Works the strictly administrative expenses to the end of September, 1913, were 1.12 per cent of the total disbursements to that date, but in addition, advertising and the fees of special counsel and commissioners of appraisal to the end of 1912, all in connection with the acquisition of land, amounted to $2,966,000, equivalent to 3.39 per cent of the total disbursements to that date, making the total for general expenses, exclusive of police services, 4.51 per cent.

On the Boston subways the general expenses, as given by the accounts, have amounted to 3.42 per cent of the total, but some items usually classed as general expenses were charged directly to the various sections of the work.

It should be remembered that in all the cases above cited, a part of the general expenses have been paid out of general funds and are not included in the above percentages.

In modern works in populated sections of the country there is a strong tendency toward an increase in general expenses, owing to the greater attention paid to policing and sanitation where large bodies of men are employed.

On the Metropolitan Water Works the charge for police services amounted to $211,000 for works costing $26,736,000, equal to 0.79 per cent.

On the New York Water Works to the end of 1912, the total disbursements amounted to $7,551,000, of which $1,369,000 was for police, equal to 1.56 per cent of the total.

The committee believes that original conditions should be considered in determining the proper percentage for general expenses and that the allowance above suggested for police and sanitation should be included only when similar expenditures were actually made in the creation of the property under consideration.

Contingencies.—In making an estimate of the cost of a projected undertaking, the experienced engineer adopts a policy of liberality with the intention of reaching the probable actual cost of the proposed work, and even under such circumstances the actual cost is as likely to exceed as to run below the estimate.

This policy of liberality includes an addition to the computed theoretical or geometrical quantities in all cases where the actual quantities are likely to be greater, the adoption of liberal rather than minimum prices for the various items of work, and a further allowance for contingencies. If the plans are incomplete, so that many minor features are omitted, he properly adds more for omissions and contingencies than where the plans are in greater detail.

In the valuation of a public service property, the same ideas should be kept in view and the percentage or sum to be allowed for contingencies should be governed, to a considerable extent, by the completeness of the inventory and the amount already allowed for omissions, by the extent to which additions have been made to the computed theoretical quantities and by the degree of liberality of the prices affixed to the various items of the inventory, but in no case should the contingencies be omitted or reduced to a small figure. Large contingent expenses are necessarily incurred in practically all important public works. They may occur from very many causes, among which may be enumerated the failure of contractors and the cost and legal expenses incident thereto; to the delay of certain parts of the work caused by such failure; to injunctions or to the inability to obtain possession of land in due season, thereby necessitationg the execution of such portions of the work under winter conditions or other adverse circumstances; to stringencies in the money market, causing a temporary shortage of funds, and a consequent disorganization of the forces employed on the work; to protracted strikes; to the necessity of rebuilding parts of the work which have failed because of improper design or unforeseen causes; to making alterations found to be necessary or desirable after the work is built, to the slipping of earth or rock, and to making changes in plans which increase the cost of work.

Such contingent expenses as those above enumerated, with the exception of the last two, should be included as contingent expenses in any valuation of an existing property because the existence of the property does not give any clew to the amount of most of the contingent expenses involved in its creation.

There is another subject closely related to contingencies which may be classed as insurance or risk. For instance, if an owner of property is constructing a building he runs the risk that it may be burned. If a fire occurs when the building is nearly finished and the owner has to rebuild it, the cost of the completed structure, if he has no insurance, will be nearly that of two buildings. In estimating the reproduction cost of the building, only one building would appear on the inventory, but in estimating its value, there should be added to the reproduction cost of the single building as otherwise determined, the sum necessary to insure its whole value against fire, and this sum should be added whether the owner actually paid it to an insurance company for carrying the risk or whether he assumed the risk himself. Similarly any other property which involves risk during its construction and testing should have added a contingent sum representing what the cost of insurance would be were there insurance companies to insure against such risks.

It is seldom that a large public service property is examined that there is not a disclosure of some large expenditure for works which have been destroyed, reconstructed, or not used because of faulty design or construction. In some cases this is distinctly the result of negligence, but in a majority of cases such expenditures have taken place where the owners were not negligent in that they have taken due care in the selection of engineers to design and contractors to construct the works. They are the result of human fallibility.

Many examples of failures have been furnished by masonry dams. A considerable portion of the Quebec bridge failed while in process of construction; there was a great slip in the Necaxa Dam, one of the highest earth dams ever built, when it was far advanced toward completion; the Loetschberg tunnel in Switzerland encountered bad ground, which required the abandoning of the tunnel for about a mile and its relocation through another portion of the mountain; the change in plan of the new Croton Dam of the New York Water Works involved an additional expense of more than $1,000,000 for construction and interest. These are only a few instances of many which might be cited. Such disastrous occurrences are not contemplated by engineers when they make a provision for contingencies in preliminary estimates of the cost of works, but it seems proper in the valuation of an existing property which has been completed and successfully tested to recognize that the owner has been required to assume the risk of accident and failure and should be compensated therefor by at least the amount which insurance companies would charge for taking such risks were they doing this kind of business. This feature may properly be included in the valuation by increasing the amount allowed for contingencies or for contingencies and risks.

Interest and Taxes.—Interest upon the capital invested in the plant up to the time when it is first operated and begins to have earning capacity is an unavoidable expense.

Under the most favorable conditions it is necessary to raise the money required for the construction of the work months in advance of its expenditure and in many cases the whole amount must be raised before beginning the work in order to insure against a suspension of operations with the large loss necessarily incident thereto.

The rate allowed for interest during construction should be the prevailing rate at the time of the valuation, having regard, however, to the variations in rate in different localities and the character of the property. The rate of interest on money required for the original plant will generally be larger than on that required for extensions and betterments as the corporation naturally has a stronger financial standing after it possesses a successful operating plant.

The amount to be allowed for taxes during construction must be determined largely on the basis of local tax rates and other local conditions and in this case, as in others relating to overhead charges, the original plant and subsequent additions should be treated separately.

DEVELOPMENT EXPENSES.

The committee has used the term "Development Expenses" to cover the investment, after the original plant and from time to time the extensions of the plant are first operated, required to equal the difference between the actual return and a fair return on the fair value of the property.

The term "Development Expenses" as above defined is substantially synonymous with the term "Going Value" when the latter is defined, as it has been, as "the investment necessary to put the plant into successful operation and to create revenues that justify its construction," but "Going Value" has been given very different definitions by the courts and others which make it inadvisable to use the term.

Good policy, both from the standpoint of the corporation and the rate-payer, requires reasonable rates for the service rendered during the first few years of the operation of the original plant, although as a result there be a deficiency representing the difference between the amount actually earned and the amount which the corporation is entitled to receive under the basic principle that it is entitled to a fair return on the fair value of its property. This deficiency must, in justice to the corporation, be made up. The fairest, most practical and definite way of doing this is by including this accumulated deficiency of earnings in the cost of the property.

This method of treating development expenses is in accord with the decisions of some public service commissions which have carefully studied the question and it is upheld by many decisions of the courts, although not in the most authoritative manner.

The above discussion relates to the original plant, which often is only a small part of the whole property. The same principles apply to additions to the plant, but the development expenses in connection with ordinary additions are very much less than those relating to the original plant.

After the property has begun to earn a proper return on the investment, minor additions can be made from time to time without interfering with a proper return on the whole investment so that there would be no charge to development expenses in connection with such minor additions.

On the other hand, extensions are frequently of such great magnitude that it is not feasible for a time to earn a proper return on the increased valuation due to them and development expenses accrue which should be treated as in the case of the original plant.

Development Expenses of Losing Venture.—It is sometimes argued that the method above described for determining the amount of development expenses is defective because when applied to a property that has not been able to earn a fair return upon what would with a normal plant be a fair value of its property, the amount allowed for development expenses would not only be much greater than for a normal property, but would continue to increase without limit.

The committee has already suggested that many principles and methods applicable to a normal property are not applicable to a "losing venture." For instance, the "losing venture" is not entitled to what would otherwise be a fair return on the fair value of the property. It has to be recognized in such a case that the investor must accept the resulting losses and such rates as the services rendered are worth, and that the actual value of the property is necessarily based on the resulting earnings and not on other methods of valuation.

WORKING CAPITAL.

Working capital is essential to the proper operation of a public service property and should be included in the valuation upon which fair returns are to be based. Any income upon the working capital coming from independent sources, such as bank interest, should be allowed for.

(To be continued.)

Simplified Formulas for Computing Car Clearances on Curves.

Contributed by George Paaswell, C. E., Asst. Engineer, Public Service Commission, New York City.

The formulas for car clearances which were given in the article entitled "Methods of Determining Standard Cross-Section and of Fixing Length of Spirals for the Main Street Subway, Cambridge," published in ENGINEERING AND CONTRACTING Dec. 10, 1913, may be greatly simplified by approximations which give sufficiently exact results even for subsurface structures.

In a circle of radius R the offset from a tangent to any point on the curve a distance x from the point of tangency is

$$\frac{x^2}{2R}$$

a formula derived from the exact expression by expansion and abandoning of the higher powers.

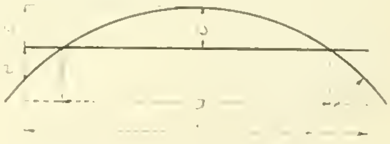


Fig. 1. Diagram Illustrating Measurements Used in Computing Car Clearances.

In Fig. 1 let a = distance between centers of trucks, let l = length of car; c = center excess, and e = end excess (this may be considered equal to the true radial excess with no loss of exactness).

From the formula $\frac{x^2}{2R}$ we get $c = \frac{a^2}{8R}$, and $e = \frac{l^2 - a^2}{8R}$, hence when a given car is specified these formulas become $c = A/R$ and $e = B/R$ where A and B are constants standing respectively for $\frac{a^2}{8}$ and $\frac{l^2 - a^2}{8}$.

Thus if one of the proposed New York subway cars is taken in which a = 17 ft. 6 ins. and l = 66 ft., the formulas become $c = \frac{282}{R}$ and $e = \frac{262}{R}$, from which values can easily be tabulated or a series of curves drawn.

If the degree of a curve is used in place of the radius we have, since $R = 5730/D$,

$$c = \frac{a^2}{8 \times 5730} D = kD$$

$$e = \frac{l^2 - a^2}{8 \times 5730} D = mD$$

where k and m are constants depending on the type of car used.

Thus, if the above mentioned car is used, $c = .019D$ and $e = .011D$. Needless to state the slide rule gives sufficiently exact results.

One interesting fact may be noticed in using these formulas. When a turnout is made from a tangent it is important to know at what point the car on the turnout no longer governs the clearance on the tangent.

Let x = distance from P. C. to point required. Then the offset of the turnout at the point is $\frac{x^2}{2R}$ and the end excess on the curve is $\frac{l^2 - a^2}{8R}$. Equating these two expressions, $x = \frac{1}{2}\sqrt{(l^2 - a^2)}$, i. e. an expression independent of the radius; hence, once the type of car is given, no matter what the radius of turnout is, at the constant distance x from the P. S. the standard clearance on tangent may be resumed.

In addition to the clearance due to center excess, is, of course, the clearance due to the tipping of the car. In New York Subway work a maximum superelevation of 5½ ins. is imposed. On radii demanding a greater superelevation than this it is assumed that the train velocities will be reduced. It is also assumed that in embanking the track the base of rail at the center of track is not changed and that one rail is lowered and the other is raised by the same amount.

From "Mechanics" taking the gage as standard, i.e. 4.7083 ft., the superelevation s required for a given velocity v is $s = \frac{.146v^2}{R}$. When maximum velocity is specified $s = c/R$, where c is a constant equal to $.146V^2$. The additional clearance due to superelevation is then, by similar triangles, $= Ks$; where K is the ratio of the distance from the governing point on the car, to the gage. Thus the total clearance due to both center excess and superelevation may be put, $C = A/R$, A being a constant equal to the sum of the above-mentioned constants.

While a theoretical expression for the length of transition required is valuable, so many other considerations enter into the selection of a proper length of transition in subway alignment: Nearness of a station to the curve; encroachment on private property; excessive excavation required by a theoretical length, etc., that usually a standard length of transition is adopted for all radii. This length is then used unless other controlling conditions determine the exact length of easement.

The following are approximate formulas for clearances required on transition curves and like the preceding formulas give sufficiently exact results for all purposes.

For center excess a distance y from the P. T. C., the radius of curvature of the transition at the point may be taken, and the clearance due to that radius on a simple curve used. Since the radius of curvature at a distance y is,

$r = \frac{L}{y} R$ where r = radius of curvature at y; L = length of transition, and R = radius of circular curve.

Using the value for center excess as obtained above, $e = \frac{a^2}{8r}$, and substituting for r its value $e = \frac{a^2 y}{8LR} = my$, m being a constant equal to $a^2/8LR$ for any given car and transition.

Likewise for excess due to superelevation the excess may be assumed equal to the excess required for the radius of curve at the point. Thus, for center excess it is only necessary to replace the R of the formula for a circular curve by a variable r which is equal to LR/y in the equation

$$C = A/R.$$

For end excess the usual procedure is to take the clearance at any point as that due to the clearance required by the radius of curve one-half car length ahead of the point. Thus the R in $e = \frac{l^2 - a^2}{R}$ is replaced by $r = \frac{L}{y + \frac{1}{2}l} R$, and the expression becomes

$$e = \frac{(y + \frac{1}{2}l)\quad(l^2 - a^2)}{LR}$$

Once a type of car is fixed, these values can all be tabulated or a series of curves drawn.

Methods for Laying Out Spiral or Easement Curves by Means of Offsets from Long Chord.

Modern practice in surveying railway easement curves, as well as circular curves, favors the method of deflection angles and uniform chord measurements wherever practicable. Probably the chief reason for this preference is, that since the circular curves are nearly always most readily staked out by that method, the easement curves at the ends of the main circular curves can generally be laid out more rapidly and satisfactorily without changing methods in performing the field work. Another reason is, that the angular deflection and chord measurement method makes practicable the use of printed tables applying to most cases that generally occur in ordinary practice. However, certain conditions may be encountered requiring resort to other methods, and railway engineers, especially those engaged in field work, should be informed as to such methods in order to be prepared to apply the one most suited to the case at hand. One such method that offers more rapid and accurate instrumental work, and which does not decrease the accuracy of the chaining, as compared to the usual method, makes use of offsets from the long chord of the spiral. A description of the long chord method, in which its application to various spiral problems is explained, is given in a paper recently presented before a meeting of The Canadian Society of Civil Engineers by Mr. E. S. M. Lovelace, a member of the society. As the method is entirely new and is in fact much needed for certain conditions of field work, we publish the paper in full, as follows:

The following method of staking out spirals was worked out by the writer when division engineer on the Quebec & Saguenay Ry., and such good results were obtained by its use, that it may be of interest to lay it before the members of the society.

The location of this railway, on the section now under construction between Quebec and Murray Bay, for the greater part of its length follows closely the north shore of the St. Lawrence River, an almost absolutely level line being obtainable at the expense naturally of a great deal of curvature. There are a couple of tunnels, and some of the smaller inlets have been crossed on rock filled cribwork, but for the most part a bench to carry the line has been blasted out of the rock. The rock cliffs on the shore side rise almost vertically to heights ranging from 100 to 1,000 ft., and to reduce the cost of the work curves running from 1° to 10° have been used liberally, the connecting tangents being necessarily of no great length.

The profile of the natural surface on the center line therefore presented the appearance of a comb with very long teeth and the cross-sectioning and staking out of the work for the contractors was in consequence very arduous and at spots dangerous. The difficulty was largely added to owing to the tides, there being a rise and fall of about 20 ft. in this vicinity. This meant that at some places, only when the tide was out, or, for about a couple of hours each day, could any work be even attempted.

Almost invariably the beginning of spiral was found to fall in a hollow, behind some large boulder, half way up a slope, or at some other equally undesirable point, and immediately in front, perhaps, would be a pinnacle of rock. In order to continue the spiral, a hub would have to be placed on top of this pinnacle and the whole spiral then swung from a backsight of say less than 30 ft. Add a couple more intermediate hubs in order to reach the end of the spiral (these were scarcely ever over 200 ft. in length) and it will be seen that, using the method of deflection angles, a considerable amount of field calculations would be necessary and then, in order to get results even approximately correct, careful chaining and most accurate instrumental work would be required; and in the writer's experience neither the one nor the other is the easiest thing in the world to obtain.

The above facts are only mentioned to show the difficulty under such conditions attending the staking out of spirals in the ordinary way by deflection angles and the method about to be described suggested itself to, and was put in shape by, the writer to overcome some of such difficulties. Afterwards it was seen that the method was capable of enlargement and applicable to other cases not at first foreseen.

By using the long chord of the spiral, the instrumental work, at least, at once greatly simplified itself, for, the angle CAP (see Fig. 1) being turned off in the first instance, it was only a question of prolonging a straight line to the B.C.C. (beginning of circular curve).

Usually in a case such as described above, where the first hub is only a short distance from the B.S. it would be found possible to place also a reference mark forward or to the rear on the same line, and such mark, in place of the short back sight to the B.S., would be used for the further prolongation of the chord.

It may at once be stated that the writer does not for a moment advocate this in all cases, to the exclusion of other methods. On unbroken ground the spirals can be put in accurately enough and perhaps more expeditiously in the ordinary way, using deflection angles, than by any other method, but where the surface is very rough (requiring intermediate hubs), the experience of the writer is that the work is greatly simplified if the staking out of the spiral be done by offsets measured from the long chord. In a trial location, through wooded country, too, the advantage of this method is at once apparent, as it is only necessary to set off and cut out the long chord of the spiral, the level rodman taking a shot to the right or left of the chord, should the ground require it. Usually, it will only be necessary to take such readings at the middle point of the spiral, which is always at a distance from the long chord of 1½ times the shift, H, the shift being the distance which the main curve has been thrown back from the tangent.

The formulæ (1) to (5) below were submitted by the writer in a paper on Transition Curves read before the society on Dec. 7, 1899, and as most of them are familiar and found in one form or another in other discussions, the writer does not propose to deal further with them here.

(1) θ (in minutes) $= 573\frac{L}{R}$ · (2) $H = \frac{L^2}{24R}$

(3) $AB = \frac{L}{2} - \frac{5}{2}\frac{H^2}{L}$ · (4) $C = L - \frac{32}{S}\frac{H^2}{L}$

(5) $L = C + \frac{32}{S}\frac{H^2}{C}$ · (6) $h = 4HS\frac{(L^2 - S^2)}{L^3}$

(7) $h_n = 4HN\frac{(A^2 - N^2)}{A^3}$

(8) $BC = (R + H)\tan.\frac{I}{2}$ · (9) $PN = 4H$ (nearly).

where R = radius of main curve; H = shift; L = length of spiral C = chord of spiral; θ = deflection angle in minutes to B.C.C.; S = any distance from A along spiral; h = offset from long chord to point distant S from A; and a = number of chords in spiral.

Number (6) however, $h = 4HS\frac{(L^2 - S^2)}{L^3}$, to be used in connection with the method under discussion, is what the writer would particularly like to draw attention to. In it, given the shift H of the main curve from the original tangent and the total length L of the spiral, then h (the offset from the long chord to a point at a distance S along the spiral from the B.S.) can be calculated.

It is a maximum when

$S = \frac{L}{\sqrt{3}}$ its value, $h_{max.}$ being $\frac{8}{9} H\sqrt{3} = 1.54\ H$; when $S = \frac{L}{2}$, $h = \frac{3}{2} H = 1.5\ H$, that is the offset from long chord to center of spiral is always $\frac{3}{2}$ of the shift.

For use in the field formula (6) can be thrown into the form given in (7) $h_n = 4\ H\ N \frac{(A^2-N^2)}{A^3}$ where the spiral is supposed divided into any number of chords of equal length. As spirals are not usually of a greater length than 300 ft., it will not often be necessary to use more than eight, which would give chords of about 40 ft. in length, but of course when required, any number (a) of chords, odd or even, may be used, and, after substitution in (7), the results worked out in the manner indicated

In most instances, however, using spirals of 200 ft. and under, four chords, or a = 4, will

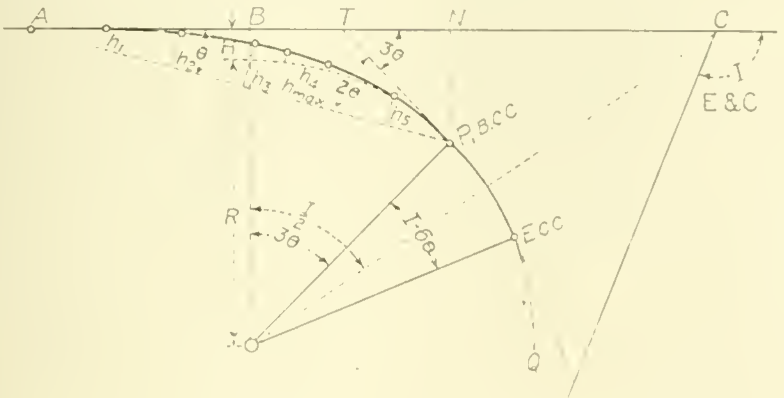


Fig. 1. Diagram Illustrating Method of Laying Cut Spirals by Offsets from Long Chord.

be found sufficient, and then the offset to the end of the first chord will always be $\frac{15}{16}$ H, to the end of the second or middle point of the spiral $\frac{3}{2}$ H, and to the end of the third $\frac{21}{16}$ H.

Using six and eight chords the following results can be tabulated (expressed decimally perhaps to simplify field computations) and marked in the field book for reference

$$\text{For } a = 4,\ h_1 = \frac{15}{16} H;\ h_2 = \frac{3}{2} H;\ h_3 = \frac{21}{16} H,\ h_4 = 0.$$

$$\text{For } a = 6,\ h_1 = \frac{35}{54} H;\ h_2 = \frac{32}{27} H;\ h_3 = \frac{3}{2} H;\ h_4 = \frac{40}{27} H;\ h_5 = \frac{55}{54} H,\ h_6 = 0.$$

$$\text{For } a = 8,\ h_1 = \frac{63}{128} H;\ h_2 = \frac{15}{16} H,\ h_3 = \frac{165}{128} H,\ h_4 = \frac{3}{2} H,\ h_5 = \frac{195}{128} H,\ h_6 = \frac{21}{16} H,\ h_7 = \frac{105}{128} H,\ h_8 = 0$$

A word with regard to the actual field work, for in the experience of the writer, engineers (especially young ones), so often use bad judgment, and choose in practice, so far as the obtaining of accurate results is concerned, the poorer of two ways of doing a thing.

For instance, while of course it would be alright to chain along the chord if only the H.C.C. is to be established (as when going through bush), it would be very bad practice indeed to do so when staking out intermediate points on the spiral, for, quite apart from the actual difference in length of the arc and its chord (be the same small or great), there would be involved the question of measuring the offsets exactly at right angles to the chord, a thing tedious and very difficult to do even approximately in the field. The proper way is to chain (using the chord length given) along the spiral, from point to point in the ordinary way.

The rodman alone moves along the long chord holding (to steady it as the measurement is taken) the zero end of the tape against a rod, the front chainman with chain tightly drawn finding the correct position of the point by reading off on the tape the offset called out to him by the man at the instrument.

The transitman sights to the side of the rod or, what comes to the same thing, the exact end of the brass ring of the tape, the rodman moving this end up or down on the rod so as to keep it approximately level with the end held by the chainman.

It is only necessary for the rodman to keep somewhere near the position where a perpendicular dropped from the point being staked out would hit the long chord, for the effect on the measured offset of his being a few inches one way or the other would be inappreciable.

In giving final track centers, the position of the stake is first found, the stake driven, and then the exact position of the tack on the stake can be found, by taking a fresh sight and measurement.

In this way it will be seen that it is feasible to stake out points on the spiral easily with no greater error than, say, ⅛ in. On the other hand, if the attempt be made to put in the spiral in the ordinary way by deflection angles, any instrument man knows that it is impossible even with the most careful manipulation to set off the angles and be sure that there is not at least an error of, say ½ in. While, of course, it may be argued that this is close enough for the most exacting practice, still the fact remains that of the two, that of offsetting from the long chord as outlined above is the more accurate method

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It frequently happens that the end of a viaduct or trestle is on a spiral and then the accuracy and ease with which points on the spiral could again and again be laid down in exactly the same position as the work progresses, would tend to recommend the use in such a case of this method of offsetting from the long chord.

For the bridge engineer too, in planning his work, formula (6) affords a ready means of finding the position and value of the maximum ordinate from the chord to the spiral, and the value of the ordinate also to any intermediate position on the spiral that it may be desired to know.

For electric tramways, etc., where the spirals are short, the above method and conclusions apply, the only difference being that it might be more convenient at time to use, in place of a transit, a piece of twine actually stretched to mark out the long chord, the offsets to points on the spiral being measured directly from this line.

When locating on side hill, or when, for one reason or another a line has to be very closely fitted to the ground, a problem frequently occurring is that case where a fixed tangent has to be connected with a curve already established. Should a simple curve answer the purpose well and good; but it will often happen that the requirements necessitate the use of a compound curve, and in that event, a simpler solution, the writer has found, is to at once employ, in place of the compound, one long, continuous spiral. While the elements of such a spiral could doubtless be worked out using the ordinary methods, yet nevertheless, the solution is given here, as the question of setting out the spiral simplifies itself to some extent by using offsets from the long chord.

Referring to Fig. 1 and the formulæ; suppose AC the fixed tangent and PQ the established curve.

Find T, the point of intersection of the tangent to the curve at P with AC.

Measure PT and the angle PTN = 3θ

In the triangle PTA, the side PT and angles PAT = θ and TPA = 2θ are known and therefore the sides of PA and TA can be calculated.

Also, NP = PT Sin PTN = 411 (nearly) is known.

Use ¼ NP as a value for H in formula (5), C being the side PA, and get the length L of the spiral.

Knowing θ and L, the radius of curvature of the spiral at P is given by (1) $r = \frac{573\ L}{\theta}$, θ being expressed in minutes.

An accurate value of H can then be found from

$$(2)\quad H = \frac{L^2}{24\ r}$$

TA is measured off along the tangent and A becomes the B.S. The spiral is then staked out as described above by placing the instrument at A or P and measuring the offsets from the long chord AP.

It should be noted that r, the radius of curvature of the spiral at P (supposing P has been chosen at random on the ground), is not necessarily equal in length to R the radius of the main curve; neither is it essential that it should be so, for the point P can then be regarded simply as a point of compound curvature.

If thought necessary, however, the position of the point on the established curve where r would equal R could be found by using the work done as described above upon which to base a new calculation, for

$NP + R \cos PTN - R$ would give the value of H for the required spiral

then $L = \sqrt{24\ RH}$ by (2)

and $\theta = \frac{573 L}{R}$ by (1)

therefore the point required on the established curve would be found forward or backward from P according as 3θ is greater or less than PTN the actual distance on this main circular curve corresponding to a central angle of (3θ − PTN).

Precisely the same problem as the above occurs at times when giving final track centers on construction. In such cases it is usually the result of incorrect measurements or inaccuracies in the instrumental work which have crept in during the progress of the work.

Of course it might be argued that such errors should not have been made, but the fact remains that to a greater or lesser extent they are always present and must therefore be considered.

What happens frequently is this: the points A and P on the diagram; that is, the beginning and end of the spiral have perhaps been referenced at the time the location was made and therefore at a time when the natural surface of the ground was in quite a different condition from its present condition.

When track centers are being given these points are again run in, but are found now not to have the same relative position to one another that they had originally.

The point A has perhaps moved a couple of inches to one side of its original position, and the point P an inch or two to the other, so that when the transit is set up at A and sighted to P the actual angle between the tangent and P does not agree with the original angle Θ. The discrepancy may be small and negligible, or it may amount to several minutes, in which case some adjustment will be found necessary.

Very often the easiest way to make this adjustment (more especially when the main curve is already lined up and its position therefore fixed) is to proceed in the manner indicated above, that is set up at P, run in and measure the line PT, measure the angle PTN, which is three times the new spiral angle, calculate the distance TA, measure it out along the tangent and thus get the new position of the B.S. The spiral can then be run in either by deflection angles or by offsets from the long chord. TP will, of course, be a tangent to both spiral and main curve at P, but as previously mentioned the radii of spiral and main curve at P will not of necessity be exactly equal.

Roads and Streets

The Economics of Macadam Versus Earth Road Construction.

Contributed by J. E. Tupper, City Engineer, Pomeroy, Wash.

In all engineering projects, and more especially in those having to do with public highways, local conditions are to a very large extent controlling factors. Sometimes those conditions are favorable and serve to make the problem an easier one; sometimes they are unfavorable and must be modified so as to allow of the carrying out of the plans, or the plans must be changed to conform to the conditions; and sometimes they are so uncompromisingly adverse that the only thing that can be done is to meet them squarely and overcome them. In any case it is never safe to ignore them. Many an engineering project has come to grief through failure to observe this simple principle. In this article, as indeed in most engineering articles, the locality from which it is written and which the writer had in mind must be taken into consideration if a fair interpretation of the treatment of the topic is to be had.

CONDITIONS AFFECTING ROAD CONSTRUCTION AND MAINTENANCE IN THE ARID REGIONS OF THE NORTHWEST.

The conditions in the mind of the writer are those that obtain generally throughout the arid and semi-arid regions of the Northwest, where the average annual rainfall ranges between 6 and 24 ins. and is practically confined to the winter months. The long dry season, extending sometimes from April 1 to Dec. 1, and almost invariably covering the months of July, August and September, is hard on any road, and especially hard on an earth road. The road surface that does not grind up into dust under the burden of even moderate traffic during such a long period of dry weather must be of excellent quality. Unless water is available for sprinkling and the traffic is of sufficient volume to warrant the expense the formation of dust is unavoidable on anything less expensive than an oiled macadam, and the only course left is to so construct the road as to obviate the nuisance to the greatest possible extent.

The general character of this entire region is mountainous, even the plains being cut by deep gorges with precipitous sides, rendering road location along reasonable grade lines exceedingly difficult, and when the fact is kept in mind that most of the existing roads were located without engineering assistance it is readily seen that the proper relocation of the roads is one of the most serious problems confronting us today. As an instance in point the writer has lately had occasion to relocate a main thoroughfare for a distance of 17 miles measured along the old line. The object of the relocation was to get the road on the best location obtainable, and reduce the gradients to a 5 per cent maximum. In order to secure this result it was found necessary to increase the total distance between terminals nearly 3 miles and reject all but 5 miles of the old location.

Still another feature of the conditions existing in this region is the scarcity of residences along the line of the main country roads. The land holdings are usually large and the residences are consequently far apart; but in addition to this feature they are usually located far back from the road, often entirely out of sight, and the land owner has a correspondingly less incentive for keeping the road through his land in modern repair.

PIONEER METHODS OF HIGHWAY LOCATION IN THE NORTHWEST.

Most of the main roads throughout this entire region were located when the country was new, when the money for construction was scarce and hard to get, and when by reason of these facts it was essential, in order to secure any roads at all, that the location should follow the lines of least resistance from the construction point of view. If by going over a bad hill a worse rock cut could be avoided the road went over the hill. If by frequent fordings of a stream the road could be made to follow the flat bottom of a valley and avoid the cost of sidehill construction, then the valley route was chosen. The locators cannot be blamed for this. They undoubtedly did the best they could with the means at their command, and in most cases did all that the economic value of the road at that time would warrant. It was not thought needful in those days to call in expert engineering advice. Up to a very few years ago in the State of Washington the court appointed three citizens to view out and locate a new road. If they thought best they could call in a surveyor to take the bearings and lengths of the lines selected by them, and afterward prepare and file with the court a map and field notes of the road as surveyed. When he had done this his duty with regard to the location of the road ended.

Under such conditions the introduction of poor locations and steep grades was unavoidable. Ten per cent grades were common and considered good, and grades up to 25 per cent were not uncommon.

It has been difficult to convince the people that lesser grades were practically attainable. When the Permanent Road law of the State of Washington was passed three years ago they were only able to secure the 5 per cent limiting clause by inserting the saving proviso that where a 5 per cent grade was impracticable grades up to 10 per cent might be used with the approval of the state engineers.

IMPORTANCE OF MATERIAL CONSTITUTING THE ROADBED.

The factor that is of paramount importance in a road is the material constituting the roadbed. Unless this is good the road cannot be made satisfactory. We have in this region a very great variety of materials, very few of which are suitable for a roadbed. We encounter in our locations drifting sand, coarse sand from ancient river beds, sandy loam, alkali beds that look like powdered chalk and are about as suitable for road construction, alluvial loam, clay loam, more or less pure clay generally mixed with disintegrated rock, beds of wash gravel and of cemented gravel, shell rock deposits and ledge rock. Nor is the diversity of the material the worst feature. The continuity of any one variety is so limited that several materials are usually to be met with in any one continuous mile of road. With a change of material comes of necessity a change in the method of treatment, making the satisfactory handling of either construction or maintenance very difficult.

DEFINITION OF TERMS USED.

It is necessary further to define some of the terms to be used in order that there may be no confusion in the minds of the reader. Macadam road construction has been a growth and so many changes have been introduced since the time of MacAdam that the term has come to convey a vastly different meaning and to cover a great variety of construction. I shall limit the term to roads where the subgrades have been properly constructed, drained and compacted, and the surface covered with broken stone to a uniform thickness, a suitable binder introduced, and the whole rolled with a heavy road roller until the surface has become firm and compact. As the term is still so comprehensive as to include several of the patented constructions I will further limit it to the so-called waterbound construction.

Under the term earth roads I intend to include all forms of construction, or lack of construction, below the grade of macadam by which a trail or road is secured over which animals and vehicles can pass. Again sub-dividing into ordinary and improved earth roads, I shall class as improved all those where a reasonable attempt has been made to secure a scientific location, with a low and fairly uniform gradient, and where a fair amount of intelligent effort has been expended in correcting or overcoming the adverse local natural conditions to the end that a road has been secured over which a reasonable load can be hauled without an undue expenditure of energy at most seasons of the year.

BASIS OF COMPARISON.

With such a classification it is evident that while the question of economy rests between the improved earth and the macadam construction, for the purpose of comparison both must be referred to the basis of the ordinary earth road. While the handicaps with which travel has to contend on earth roads are so many and various that it is difficult to establish an average condition, we will have to assume that the conditions are such that a reasonable load may be hauled, and that no grades that are so steep as to be prohibitive for loaded teams, nor such extended stretches of excessively bad road exist as to render hauling impractical. It is frequently stated that a road is no better than its poorest part, and while this seems to be good logic it is not necessarily true in all cases.

A short stretch of 10 per cent grade in a road does not reduce the value of the road to that of 10 per cent for its entire length. A team can exert twice as much energy for short distances and for short periods of time that it can exert continuously over extended periods or distances. So the condition of the ordinary earth road must be taken as that existing on the greater portion rather than that of the excessively poor short stretches.

IMPROVEMENT OF EARTH ROADS.

Except for two features that have been largely exploited during the past few years there has been very little consideration paid to the question of improvement of earth roads. Crowning and drainage, which are really one and the same thing, and the use of the split log drag have been extolled as the last word in earth road improvement. If they were insufficient then the only recourse was hard surfacing. Both of these features are of value in the maintenance of the road, but neither of them touch the real essential factor in the construction of a serviceable earth roadbed.

The two things that are the curse of earth roads are dust and mud, and we cannot have a serviceable earth road until we secure a foundation that will not break down under the traffic under adverse weather conditions. The quality or condition of the immediate surface is of less importance than the condition of the subgrade or ultimate foundation. We cannot hope to entirely eliminate the formation of mud and dust, but we can so construct the road that mud and dust will not form to such an extent as to become serious obstacles to travel. Anyone who has traveled extensively over country roads can readily recall places on the roads where dust is never troublesome even in the driest weather, and where the mud never gets deep enough to become an obstruction to travel. An investigation of conditions existing at these points will invariably show that the foundation of the roadbed is stone in some of its forms. It may be ledge rock, shell rock, broken stone, gravel or coarse sand, but it is never fine earth that is free from stone. Under ideal conditions a roadbed of loam or clay cannot be improved upon. The trouble is ideal conditions cannot be secured at all times. Sprinkling will keep down the dust in summer, but there has been no means found for preventing the formation of mud in winter. Given a sufficiently copious rainfall and a heavy enough traffic and any loam or clay surface will become a mass of mud regardless of crown, drainage, or previous use of the log drag, and if both rainfall and heavy traffic are continued long enough the only limit to the depth of the mud will be the ability of the teams to haul the wheels through, unless a stony foundation is reached before prohibitory conditions obtain.

In most places and under most conditions proper crowning of the road will largely control the mud problem, but crowning in no way restricts the formation of dust. Furthermore what value is there in crowning a road that runs through a sand bed, or over porous gravel, or has been blasted out of a rock ledge? The only reason that can be given for crowning a road is that the rain falling on the surface may run into the side ditches instead of remaining in the road to be instrumental in forming mudholes. However, unless the surface is kept free from ruts the crowning will help very little in preventing the formation of mudholes. It is here, and only here, that the log drag is efficient. In clay, "dobe," muck, loam that is subject to supersaturation, and across swampy lands there is no question but that crowning, drainage, and the use of the log drag, if intelligently planned and carried out, will materially assist in giving good roads for the greater portion of the year, but unless there is provided a foundation of stone underlying the surface such roads will cut up badly in extended spells of either dry or wet weather. Such material will not become and remain compact unless supplied with just the proper amount of moisture. If there be too little or too much the impact of the traffic will grind it into dust or mud. The only recourse, other than actual paving, is to supply the stone subgrade where it is lacking, so that when the wheels cut through the immediate surface coating they will meet with sufficient resistance to arrest further damage. There must be a sufficient thickness of the stone to bear the weight of the traffic, it must be sufficiently compact so as not readily to disintegrate under the traffic, and it may be well concealed under a surface coating of the local clay or loam. This will give the appearance and the advantages of the common earth road at most seasons of the year with the added assurance that the dust cannot get very deep nor the mud very troublesome even under the most adverse weather conditions.

There are miles of road in western Oregon that a few years ago were literally bottomless bogs for several months in every year, but which are now fairly good roads at all seasons. The change has been brought about by simply dumping enough broken stone in the road along the center line of travel to form a bed that would carry the traffic. This stone was sometimes roughly spread and sometimes left to spread itself, and the compacting was all done by the feet of the teams and the wheels of the vehicles. All this is undoubtedly very crude and unscientific, and it may be very unethical to advise or encourage such primitive methods, but the fact remains that these methods do secure desirable results. I have heard noted highway engineers inveigh against the dumping of broken stone in the mud of the road as being not only a waste of energy but an actual damage to the road so treated. From their point of view they may be right, but the practical man who is looking for service rather than frills cannot agree with them.

Fifty years ago, when but a lad in Pennsylvania, I saw my father filling mudholes in the road with shale, later on in Minnesota I helped to fill such places with gravel, and where the roads crossed "sloughs" in Minnesota and Iowa I saw them constructed of gravel or the waste from the limestone quarries. I have seen the waste from the ore concentrators in Idaho and Montana used in this manner to build roads to the mines, and not a mile from where I sit as I am writing this article, a piece of road is located that was treated to a coat of broken stone five years ago this winter at a time when the mud was 12 to 18 ins. deep. The stone was obtained at the waste pile from the excavation for one of the city reservoirs and dumped in the mud to an average depth of 12 ins., spread to a fairly uniform surface and then covered with 2 ins. of clay. This place had been an almost impassable morass every winter for years past, but since filling with stone has been as good a piece of road as we have. Another piece 600 ft. long is being treated in the same manner at the present time at a cost of 25 cts. per square yard.

It is not my intention to claim that roads treated in this manner are ideal roads. When the wheels cut through the surface coating to the stone foundation they are apt to strike rather rough going. Some of the stones may work to the surface, though I have never known this to occur where reasonable care was taken in placing the material.

The road will not be free from dust in summer nor from mud in winter, but neither dust nor mud can get deep enough to offer any serious obstacle to travel. They are not built to meet sanitary requirements or esthetic appearances, nor to compete with roads that are laundry-starched and smoothed out with hot-point electric irons.

On a very large portion of the country roads the travel is too heavy and important to be condemned to the use of unimproved earth roads, and yet not of sufficient volume to warrant the construction of hard surfaced roads. The construction herein outlined, with its flexibility and practical value, is designed to meet the requirements of just this class of traffic. The object to be aimed at is the securing of a stable foundation throughout the entire length of the road. Where the foundation is naturally good advantage is taken of the fact and where it is defective to a greater or less extent the needed reinforcement is provided. It does not mean that dumping broken stone in the mud is advisable in all cases. It may be better to remove the mud coating and spread the stone evenly on the firmer subgrade, and then cover it with the best material available. We all know that there are always a few bad places in every ordinary earth road, and if these could be eliminated the road would not be a bad one. It is here that the highway engineer must use common sense rather than follow scholastic maxims. The principle laid down by the builder of the "One Horse Shay" is a good one to apply to country road construction.

"The thing to dew is only jest
T' make the weakest place 's strong 's the rest."

There is a broad gap between the pack trail and the pavement that must be filled. How to meet the requirements of this class of road is a greater economic problem and affects more people and more traffic than the problem of the best binder for macadam.

There is nothing surer than that for years to come the great mass of country travel must pass over earth roads for part of the way at least. We cannot afford to macadamize all of our roads, and if we could it would not be a paying investment.

ECONOMIC ANALYSIS OF THE PROBLEM INVOLVED.

As an economic proposition the amount that can be profitably expended on any road is directly proportional to the amount and importance of the traffic passing over that road. Like all general principles this cannot always be applied literally. The question is one of public policy and like all such questions cannot always be reduced to the dollar and cents standard. Sanitation or esthetics may be factors of equal or even paramount importance, but when these, or any other extraneous factors enter the problem it is because of strictly local conditions and does not affect the status of the general proposition. Comparisons can only be made by accepting certain fixed rules. If we allow local disturbing conditions to enter the problem then the comparison becomes a strictly local one and of no value in determining general applications.

In judging the comparative value of different kinds of road surfaces the most convenient, and in fact about the only factor we can use is the efficiency of a team on the roads compared. Numerous attempts have been made under varying conditions to ascertain what these ratios are, and the results do not agree very closely. Haswell finds the ratio of the average all day loads to be as follows: Macadam 100 per cent, earth roads in prime condition 86 per cent, ordinary earth roads 32 per cent, and sand 16 per cent. Byrne gives the ultimate possible load as macadam 100 per cent, best earth 51 per cent, ordinary earth 21 per cent, and sand 16 per cent. Except in a general way these conclusions cannot have very much value because of the fact that it is not so much what a team can move over the roads as it is what results we may expect to obtain in actual practice. Numerous attempts have also been made to ascertain the cost per ton mile of moving freight over different kinds of roads with results that are still more conflicting. They apparently ignore the costs of loading and unloading the freight, which with farm produce is an important item, and also the fact that while one driver may handle six or eight horses on an earth road it is hardly practical for him to handle a six or eight-horse load on macadam under ordinary country conditions.

The cost of handling the produce and the driver's wages would usually be the same whether the load was drawn by two horses over a macadam road or by eight horses over an earth road. The writer has made quite extensive investigations with reference to the costs of moving farm produce in this region and the result seems to show that the average cost is 31 cts. per ton mile over ordinary earth roads, the usual team consisting of six horses, the load 4 tons exclusive of the vehicle, and the distance moved 8 miles, the team returning empty. As the driver has to handle his load in the field and again at the warehouse, besides caring for his six horses, he can hardly be expected to do the same with a 12-ton load which would be only an ordinary load for his six horses on good macadam.

If the field is so located that two horses even with the help of a snatch team can get the load on the macadam road then the cost can be reduced to perhaps 19 cts., or a saving of 15 cts. per ton mile. However, not all farms are so located that they can take full advantage of the easier haul over the macadam road even if the system were extended to the full economical limit. A considerable portion of the farms must remain so located that the initial haul will be over earth roads for perhaps several miles. The traffic over these feeders would be of too small volume to warrant hard surfacing. This would reduce the benefit that would accrue to the total tonnage passing over the road by probably one-half even with a complete system of hard surfaced roads, but on the other hand the benefit of the improved road to the lighter traffic would undoubtedly be sufficient to fully offset this loss. Assuming such to be true we can then see what volume of tonnage must be shown in order that the macadamizing of any given piece of road may be justified as an economic proposition.

SOLUTION OF A TYPICAL PROBLEM.

Assuming $10,000 as the cost of the road per mile, we have then to provide for an annual interest charge on that amount in perpetuity, a sinking fund sufficient to renew the surfacing at stated periods, and an annual maintenance charge sufficient to keep it in reasonable repair. The interest is on the total investment of $10,000, which at 4 per cent amounts to $400. The sinking fund would only provide for renewing the top course of the macadam surface at ten year intervals and for this purpose $200 per annum must be provided. For ordinary maintenance we will allow $300, which ought to be ample under any conditions where a macadam surface is economical. This gives an annual charge of $900, which we must show is derived in benefits to the traffic if we would justify the investment. At 15 cts. per ton mile as shown above this would only require 6,000 tons of traffic. This result could only be predicated where the improved road system was very complete, including not only the hard surfaced main trunk lines, but also improved earth roads as feeders. Under ordinary conditions, and while the improved road system of the community is in its formative period, we would scarcely be justified with a tonnage less than three times that, or 18,000 tons.

It is evident that as the lines of improved road are extended farther into the country and nearer the primary sources of traffic a greater proportion of the traffic will be enabled to get the full benefit if the improvement and the worth of the road as an investment will be enhanced. It may be demonstrable that under certain conditions an investment of $10,000 per mile would be justified with considerably less than the 6,000 tons of freight in sight as tributary to the road, and as where the trafic is small a narrower roadbed may be sufficient, and in many instances the cost of grading is much less, we may be able to build a sufficient road for $1,000 per mile and consider it profitable where the freight traffic is 2,000 tons. Below this limit it is not probable that macadam can be justified and some form of improved earth road must be resorted to. In most cases for comparatively light traffic a properly constructed earth road may be fully as good as macadam and very much cheaper. It takes a considerable amount of traffic to turn a compact earth surface into a dust bed, and though it will be muddy to a certain extent after every rain, unless the travel is quite heavy the mud will not get deep enough to offer serious obstructions to the travel and no bad mudholes will form if the road has been properly made. As the construction cost of an improved earth road is so largely composed of grading the cost will vary with local conditions and may be anywhere from $1,000 to $6,000 per mile.

As this is all permanent work no sinking fund need be provided and the only annual charge will be the interest on the investment. To be sure there will be maintenance charges, but as they should be no greater, but rather less, than the cost on the ordinary earth road they can hardly be charged up against the improvement.

This gives us an annual charge as low as $40 and as high as $210 to be offset in benefits to the trafic.

As the efficiency of such a road is fully equal to macadam for light travel, and is about 60 per cent that of macadam for loads, the aggregate efficiency will be fully 80 per cent if the volume of travel is not excessive. This being the case an investment of $6,000 per mile would be justified for 1000 tons of freight. The question of whether improved earth or macadam is to be preferred will usually hinge on the cost of the grading. If the cost of this is light then macadamizing might be justified for a comparatively small tonnage while an earth road would be profitable with a very high cost of grading even when the tonnage was so light that macadamizing could not be considered.

CONCLUSIONS.

The whole question turns on the annual cost of the improvement and its value as a public utility. Unless its value is at least equal to its cost then the investment is not justified.

The public is entitled to as good a road as the value of the traffic will justify, and no more. So far we have treated each mile of road as a separate entity having a value proportionate to the amount of traffic it carries regardless of the way its improvement affects the adjacent parts of the system. To a large extent this is true of the terminal of a trunk line adjacent to the trade center, but as the distance from the trade center increases the interdependence of the adjacent parts becomes more marked. Thus while the traffic over a given mile of road located 10 miles from town may not be of sufficient volume to warrant the expenditure of any great amount of money for improvement taken by itself, the fact must be borne in mind that an improvement that will allow the movement of an increased load over that mile of road will allow the movement of the same increased load over every one of the ten intermediate miles, provided they are also improved.

At the same time as the feeders of any trunk line are improved the value and utility of the trunk line is increased and the ratio of cost to value is decreased. Nor is the traffic of the present and the immediate future the sole measure of the value of any improvement. A road that may be sufficient for the present may be entirely insufficient ten years from now. Still it would be unwise to bank too much on the future. The wise construction is that which is amply sufficient for present needs and at the same time provides a foundation for future improvements. A properly constructed earth road is but the necessary foundation for a macadam road, a worn out macadam forms an excellent foundation for concrete, and either a prime macadam or concrete is essential as a foundation for asphalt.

If it is unwise to construct macadam or other hard surfaced roads before the traffic is sufficient to warrant the expenditure, it is equally unwise to continue to spend money in patching up earth roads after the traffic has become greater than they can handle. The most unwise course of all is to insist that there is no middle course between the ordinary unimproved earth road and the costly hard surfaced construction, and to refuse to consider any methods of improving the earth roads until such time as the traffic has become sufficient to warrant the better construction. There is no method of road construction so good that there is not something almost as good and a little cheaper. At the same time the best we have been able to devise as yet is still subject to improvement. There is a fairly uniform and well graduated sequence between the poorest road construction and the best, and that road is the best for any given locality which shows the most nearly perfect ratio between cost and value.

A well constructed pack trail may be as ideal a road for certain mountain conditions as a wood block pavement is for congested city traffic. One of the worst drawbacks that we have to contend with in the good roads crusade is the lack of co-operation between the advocates of the split log drag and the engineers who insist on macadam or concrete surfacing as the only solution of the problem. Each is beating out his own little tune on his own little drum regardless of the roll of the general harmony, or whether he is in time with the beat of the marching columns.

History of Bituminous Concrete Pavements in Which Broken Stone or Gravel is Used.

The history of bituminous concrete pavements is of value for record and an unusually interesting history is contained in a paper before the American Association for the Advancement of Science at its meeting at Atlanta, Ga., last December. This paper was prepared by Joseph H. Conzelman, graduate student in highway engineering, Columbia University, and is in full as follows:

Early in the nineteenth century bituminous concrete was used for the purpose of paving sidewalks, but its use as a street paving material did not begin until 1840, when a pavement of tar-coated stone was constructed at Nottingham, England. Five years later a similar mixture was used at Sheffield, England. In 1854 the use of asphalt for the carriageway of streets was commenced in Paris and in 1869 the authorities at London started to use this material. The majority of the pavements laid in Paris and London during this period were made of powdered rock asphalt and pulverized stone, iron and grit. This mixture was spread to a depth of 1½ ins. to 2½ ins., after compression, on a concrete base 6 to 9 ins. thick.

In this country rock asphalt was not as available as in Europe. Consequently our road engineers turned their efforts toward finding a cheaper substitute. Up to 1872 about one hundred patents had been issued on bituminous pavements, a few of which specified the use of a clean, hard mineral aggregate. The majority, however, to become patentable sacrificed quality, and called for the use of various percentages of such materials as sawdust, charcoal, peat, shavings, etc. A board of engineers representing many of the large eastern cities reported in 1872, in part, as follows:

> A pavement composed of broken stone, gravel, sand, or their equivalents, if they can be found, united with asphalt and coal-tar bitumen, properly prepared, without doubt, will make a roadway which shall prove both cheap and durable; but no process now in use, so far as known to the members of the Board, by which pavements are at present manufactured, offers such conditions of uniformity in the manipulation of the work as to warrant assurances of substantial success.

The first bituminous concrete used in this country to pave carriageways of roads and streets, to the best of the writer's knowledge, was laid in New York City by the Scrimshaw Paving Co. in 1861. The process used was patented in 1860 and consisted of successive layers of well rolled mixtures of broken stone, gravel, sand, coal ashes, cinders and coal-tar. The coal-tar was not refined and the pavement could stand but little traffic before the light oils had evaporated. About 1869 this company was offering to keep any pavements, built by them, in repair for ten years at an annual charge of 10 cts. a square yard for the first five years, and 15 cts. a square yard for the following five.

It soon became apparent that a mixture must be used which would harden quickly. The Fisk concrete was the first experiment in this direction, resin and pine pitch being the hardening agents used. A sample of this pavement was laid on Fifth Ave., New York, about 1869 and for the first few months proved very satisfactory. As soon, however, as the lighter oils had entirely evaporated, the pavement became so hard and brittle that it cracked and powdered under traffic and the avenue became a mud hole in rainy weather. This gave rise to the cry of "Poultice Pavements" and did much to prejudice the country against the use of tar as a road material.

About this time, 1871, Mr. N. B. Abbott, after considerable experimenting with coal tar, secured a patent and laid a sample mile of pavement in Brooklyn on Sackett and Degraw streets. The bituminous material used consisted of a mixture of coal-tar pitch and creosote oil, and for the mineral aggregate only the hardest broken stone and gravel obtainable were used. All of the material was heated before mixing and was laid and rolled while still hot. The pavement was laid in two courses varying in thickness from 2 to 5 ins. for the bottom layer and from 1 to 3 ins. for the wearing surface.

Mr. Abbott heated and mixed this material by machinery propelled either by steam or horse power; used a revolving screen for his stone and gravel; and considered rolling very essential. His pavements cost from $2.50 to $3.00 a square yard. A coating of from 2 to 3 ins. laid over an old cobblestone, macadam, or other stone or wood block pavement varied in cost from $1 to $1.50 per square yard.

In 1873 Mr. Abbott patented a flush coat process which he called the Abbott Grit Surface. This invention consisted in applying to the wearing surface of a pavement a coating of bituminous cement sprinkled with clean, dry grit or sand. He claimed that by applying this coating periodically the life of a pavement would be prolonged and slipperiness prevented.

Of these early day pavements the one which most closely approximates the present idea of good bituminous concrete construction was the Excelsior Pavement. This pavement is described by Mr. Gabriel Leverich in a pamphlet printed in 1871. Mr. Leverich says in part:

> Any pavement held together by bituminous cement, from which the oils, volatile at ordinary temperatures, have not been expelled must ultimately fail.
>
> For a cement, a stable and intimately united compound, one which does not change on exposure, should therefore be selected. This done in time, and under exceptional and obscure circumstances, forms a strong and elastic cement, which may be obtained at once by a proper application of heat to the tar (alone or mixed with pitch) to separate before use the lighter parts which otherwise will evaporate afterwards.

The tar was refined in a still placed over a portable boiler. Steam was circulated in and around the still and was allowed to escape through perforated pipes and mingle with the tar. The mineral aggregate is described as follows:

> The resisting material may be sand for the surface, gravel or broken stone mixed with sand for the middle, and larger stone for the bottom, or bed; broken stone alone is preferred, sizes being chosen which best form a compact structure, smooth on top, and strong throughout. Small fragements are not generally required. Several sizes are mixed to form a close mass without cavities, the coarser stone being put beneath. This material should be carefully selected for its strength and resistance, and contain no dirt or other foreign matter.

The resisting material for the covering was dried and heated in a cylinder placed over a furnace. This cylinder rotated about a slightly inclined stationary axis fitted with fixed arms for stirring. The material was mixed with heated tar in a similar cylinder.

The Excelsior Pavement has been described in a specification as follows:

> The gravel or broken stones shall be sharp, rough and hard; clean, dry and free from foreign matter. In quality, the gravel shall equal the best "Ryder" or "Peekskill," and the broken stones, that obtained from the Palisades trap rock.
>
> Broken stones are preferred for the whole pavement and shall alone be used for the covering. The greatest dimensions of stones for the base (except as hereinafter noted) shall be between 3 ins. and ¼ in., and for the covering, between 2 ins. and 1/20 in.; the sizes shall be mixed in proportion, varying with the size to form a close mass, which when dry and compacted can absorb not more than 20 per cent of water. The whole shall be completely dry when used; the stones laid in the base may be heated or not, according to circumstances. Those laid in the covering shall, as used, be uniformly heated to a temperature between [illegible] and 30[illegible].
>
> The cement shall be gas tar alone, or mixed with not more than one-third asphalt or pitch. It shall be distilled as used, in direct contact with steam—separated from the more volatile oils and uniformly heated to a temperature between 275° and 315°, whereby it is brought to a certain fixed standard, so that for the base it shall be viscid and coherent when newly laid and for the covering [illegible] at 100[illegible]; solid at 60° and tough and strong at [illegible].

The specifications call for a [illegible] to [illegible] ins. thick, of a single course of wedge-shaped stone nearly uniform in size. The crevices to be filled with smaller stone, then covered with bituminous or lime cement. An old cobblestone pavement could be used for the sub-base in which case the stones were to be cleaned to a depth of [illegible] ins. below their average surface.

On this sub-base was placed a base course varying in thickness from 4 to 6 ins. The stone for this course was covered with a "slight but entire" coating of hot base cement. Upon this was placed the wearing surface 2 to 3 ins. in depth. The materials for this course were heated and mixed in the proportion of 1 cu. yd. of stone to 20 to 25 gallons of bituminous cement. Each course was carefully rolled with a steam roller weighing at least 10 tons. Before final rolling the surface was slightly coated with fine gravel.

Bituminous concrete pavements were laid to a large extent in Washington, D. C., from 1870 to 1878. Approximately 700,000 sq. yds were put down during this period of which

20 per cent failed within 2 to 3 years.
30 per cent lasted 7 years.
41 per cent lasted 15 years.
4.9 per cent lasted 20 years.
1.2 per cent lasted 30 years.

The most permanent of these pavements was the Filbert Vulcanite. It consisted of a mixture put together in the following proportions: 80 lbs. asphalt; 30 lbs. of either resin, pitch, coal tar or pine tar; 5 lbs. of sulphuric acid; 90 lbs. of lime and 600 lbs. of sand, fine gravel, ground iron cinders and broken stone. The mineral matter was dried and mixed in proportions tending to give a close mass. The heated bituminous cement was then added and the material was spread over the subgrade to the desired thickness and rolled while warm.

Among other pavements laid in Washington during this period, were the following:

Parisen Pavement. The base course consisted of a layer, 6 to 8 ins. thick of 2 to 1 in stone thoroughly mixed with coal tar, lake asphalt, fine screened gravel and calcined gypsum. A light course of stone chips or gravel mixed with medium tar and Portland cement was then rolled into the surface voids. This coating was painted with the cement used in the base and on this was spread a 2-in. course of gravel, gypsum and Portland cement mixed with a bituminous cement, three-fourths of which was distilled tar, one-eighth Cuban asphalt and one-eight Trinidad asphalt. This course was then rolled, painted with bituminous cement, and repeated.

Scharf Pavement. This type proved more durable than many of the others, 200,000 sq. yds. of it being laid. It was put down in three courses, all well rolled. The base varied in thickness from 4 to 7 ins. and consisted of a mixture of broken stone and stiff bitumen derived from the slow distillation of coal tar. Upon the base was placed a 2-in. course of stone mixed with cement consisting of 1 gal. of pitch to ½ lb. of asphalt. The wearing surface was composed of small stones and gravel, hydrated lime, cement, pitch and asphalt

The chief defect of these old Washington pavements was in the wearing surface; the foundation rarely, if ever, failed. The results obtained, however, were so variable that in 1878 the use of bituminous concrete was abandoned and for nine years none of the material was laid.

At the end of this period some of the coal-tar pavements were in such good condition that the Engineering Department in its report for 1887 stated that it believed that if proper care were taken, such pavements could be laid cheaper than sheet asphalt and with equal success. The average annual expenditure for maintenance, during the fifteen years in which these pavements had been in use, was 7½ cts. a square yard. One of the pavements, a Vulcanite, which had been down fourteen years, had required an average expenditure of only 2.9 cts. per square yard per year.

The use of bituminous concrete in Washington started again in 1887 and during the next five years 125,000 sq. yds. were laid. The pavements used were known locally as the "Coal Tar Distillate" and the "Combination." Both were 7 ins. in thickness, having a base and binder course of 5½ ins. and a wearing surface of 1½ ins. The base consisted of 4 ins. well rolled, of stones passing a 3-in. ring. This course was then coated with coal tar paving cement, about 1 gal. to the square yard. The binder course, a mixture of 1 gal. of coal tar to 1 cu. ft. of screened stone not larger than 1¼ ins., was then spread to a depth of 2 ins. and rolled.

The wearing surface of the coal tar distillate pavement was composed of 60 per cent of sharp sand, 25 per cent screenings, 14 per cent paving cement and small quantities of hydraulic cement, slacked lime, and sulphur. The cement consisted of 30 per cent asphalt and 70 per cent coal tar. The wearing surface of the "Combination" pavement consisted of the ordinary sheet asphalt top mixture.

It will be seen that the pavements of 1871-1878 differed greatly in construction from those just described. When cut up the older pavements show a closely compacted mass, the stones of which are well coated and the few voids filled with a bright, fresh appearing tar. Those of the latter period are brittle and the base course contains but a small amount of tar, and that dead and lustreless.

After 1880 privately owned pavements decreased in popularity and unpatented types of construction were experimented with in many cities in the United States and Canada. At Hamilton, Ontario, tar concrete pavements were first laid in 1880 and have since been used extensively.

The pavements at Hamilton consisted of a base course 6 ins. thick of hand broken stone, furnace clinkers or brick, rolled smooth. Upon this was placed a 3 in. course of hand broken tarred stone passing a 2½ in. ring. This was followed by a 2-in. layer of machine broken tarred stone smoothed off with a hand roller, sprinkled with a dressing of fine material and thoroughly rolled with a steam roller. A ¾-in. layer of tarred gravel and quarry chippings was then applied and well rolled. The pavement was then finished off with a dressing of stone chippings.

Following the experience of Hamilton, the city of Toronto in 1890 laid about one-fourth mile of tar concrete. In this work a 4-in. foundation of broken limestone was used, the sizes of the stone varying from 1¾ to 3 ins. The voids were filled with sand and the course flushed with tar and rolled. A layer of tarred stone was then spread, not less than 1½ nor more than 2 ins. deep. This was evenly raked and rolled and upon it was placed a ¾-in. layer of fine granite chips thoroughly tarred and rolled. A dressing of pea gravel was then spread over the surface.

This pavement stood up well for two or three years, but began to disintegrate at the end of that time. In 1901 the specifications were changed to require a 5-in. foundation course. The intermediate course of tarred stone was eliminated, a 3-in. wearing surface being used. In 1905 the mineral aggregate of this wearing surface was graded as follows:

> 30 per cent shall be such as will pass a No. 10 sieve.
>
> 12 per cent shall be such as will be held on a No. 10 and pass a No. 8.
>
> 10 per cent shall be such as will be held on a No. 8 and pass a No. 4.
>
> 18 per cent shall be such as will be held on a No. 4 and pass a ½-in.
>
> 30 per cent shall be such as will be held on a ½-in. and pass a ¾-in.

The bituminous cement specified was a mixture of coal gas tar with a specific gravity of 1.2 and pitch having a specific gravity of 1.15 and a melting point not under 200° F. The specification also called for a flush coat of California or some other equally good asphalt, having a melting point of about 170° F.

The bituminous concrete pavements laid in Canada generally proved very satisfactory for the first three years. At the end of five years, however, the wearing surface was worn off in patches and after ten years of continuous use very little of this surface remained.

Bituminous concrete pavements were laid at Littleton, N. H., in 1894-95-96. A total of 11,000 sq. yds. of this material was put down at a cost of about $1 per square yard. These pavements were constructed in the following manner: Upon a 5-in. base course of hand set cobble stones was laid 2 ins. of 1½-in. tar coated gravel; over this a 2-in. layer of ½-in. tarred gravel, followed by a 2-in. surface coat of sand mixed with coal tar and pitch. These pavements proved very satisfactory and for eight years required very little maintenance. A similar pavement, however, laid at Barre, Vt., in 1890, subjected to heavy teaming traffic, rapidly disintegrated.

In 1891-1892 there were laid in Denver, Colorado, bituminous concrete pavements consisting of a mixture of several sizes of broken stone and an asphaltic cement. Mr. Marcus H. Hodgman, the man responsible for this work, claimed that by carefully grading the mineral aggregate a keying together of the mass was effected which gave it inherent stability independent of the cementing material.

Mr. Frederick J. Warren, at that time connected with the Barber Asphalt Co. at Denver, became convinced of the merits of a graded aggregate; and, after careful experiments, he claims to have obtained the exact proportionment of sizes which would reduce the voids in the mass to a minimum. In 1901 he obtained a patent on a paving process which consisted of mixing a bituminous cement with broken stone, the latter to be carefully graded as to size in such a way as to contain less than 21 per cent of voids.

Mr. Warren's pavement, known as "Bitulithic," was first laid in Pawtucket, R. I., in 1901. Its success was immediate and at the close of the year 1907, 6,500,000 sq. yds. or 422 miles of 30 ft. width, had been laid in cities in the United States and Canada.

Following three years of experimental work with bituminous concrete, Rhode Island in 1909 was the first state to adopt this method of construction as a standard type for use on state highways. The authorities in Rhode Island have met with very good success in using for the road metal a product known as one size crusher-run stone. The following is a mechanical analysis of a typical product used for the wearing surface:

Per cent passing 10 mesh screen	1.0
Per cent passing ¼-in. screen	2.5
Per cent passing ½-in. screen	30.8
Per cent passing ¾-in. screen	34.2
Per cent passing 1-in. screen	23.4
Per cent passing 1¼-in. screen	8.1

A product of this kind is obtained from the ordinary type of crushing plant and results from stone passing over a ½-in. to ¾-in. screen and through a 1¼-in. screen.

In 1908 there was laid at Topeka, Kan., a type of pavement similar to the ones laid by Mr. Hodgman in Denver. Before the pavement reached completion an injunction was placed on the work following charges of infringement made by the Warren Brothers Co. Evidence was obtained to show that this type of construction had been in use prior to the issue of the Warren Brothers patent and a compromise was effected. In 1910 a similar pavement was laid in Creston, Ia., and the usual injunction proceedings were instituted. The officials of the city obtained the necessary evidence and an agreed decree was entered into. The following extract is taken from a decree signed by certain city officials and the Warren Brothers Co.:

> ——and that any pavements hereafter constructed in substantial compliance with the following formula, to wit:
>
> Bitumen, from 7 per cent to 11 per cent.
>
> Mineral aggregate passing 200 mesh screen, from 5% to 11%.
>
> Mineral aggregate passing 40 mesh screen, from 18% to 30%.
>
> Mineral aggregate passing 10 mesh screen, from 25% to 55%.
>
> Mineral aggregate passing 4 mesh screen, from 8% to 22%.
>
> Mineral aggregate passing 2 mesh screen, less than 10%.
>
> Sieves to be used in the order named would not infringe the claims of said patent.

This type of construction is now known as the Topeka mix and has been used to a considerable extent with very good results.

The bituminous concrete pavements of today are too numerous for description. There are, however, two distinct classes: first, that for which the material is mixed while the construction of the pavement is in progress and, second, the prepared surface type. By the latter method the mineral aggregate is mixed with bituminous cement at the manufacturer's plant and is shipped to the user as ordered. Tarmac, a pavement of this type has been very successfully used in England. The mineral aggregate is slag, crushed and screened into sizes varying from ¼ in. to 2½ ins. The bituminous material consists of tar mixed with a small quantity of pitch, resin, and Portland cement. The lower course as generally laid consists of 1½ to 2½-in. slag laid to a depth of 2¾ ins. On this is placed a layer 1¾ in. thick of ½ to 1¼-in. slag. The road is then finished with a coating of ¼ to ½ in. dry slag screenings.

The first class, namely, those mixed while the work is being done, may be divided into three sub-classes dependent upon the character of the mineral aggregate.

(1) Those in which one size crusher run broken stone is mixed with bituminous cement.

(2) Those in which one size crusher run stone and other fine material is mixed with bituminous cement.

(3) Those in which a predetermined mineral aggregate is mixed with bituminous cement.

From this review of bituminous concrete construction, it will be seen that many of what are considered today essentials of good construction, were understood and practiced by the early builders of bituminous roads. The advantages of a flush coat and the necessity of a good foundation, clean hard angular stone, thorough rolling and a minimum of voids are not new principles. Many of the early failures undoubtedly were due to faulty design, the greater number, however, were the result of using a poor bituminous cement. It should also be remembered that at the time these pavements were first laid most city streets were paved with cobble stones and a smooth and less noisy pavement must have have attracted an excessive amount of traffic.

Method and Cost of Constructing "Top Soil" Roads in Virginia.

Contributed by A. H. Pettigrew, County Engineer, Lebanon, Russell County, Virginia.

The top soil or gray soil road differs from the sand-clay road in that the soil road is a natural mixture of sand and clay while in the sand-clay road the material is mixed when the road is built. The natural materials may have become mixed in the fields along the road by successive cultivation; it is known as "top soil" or "gray soil." This type of road is being built in a great many counties of Piedmont, Va., and is giving general satisfaction.

Organization and Equipment.—An equipment for one force consists of 16 to 20 mules, one 6-horse grader, one 2-horse grader, one rooter plow, one 4-horse turn plow, one 2-horse turn plow, about seven wheel and drag scrapers, one wagon with slat bottom for each 2-horse team, one heavy disk harrow and 1 split-log drag for every four or five miles of road to be constructed, one spike-tooth drag and small tools, such as picks, shovels, etc. An equipment of this size will cost about $6,000. Two foremen and 20 to 30 laborers are usually required to operate this equipment, and the cost per month will average about $1,500. A force of this size will build from one to three miles of soil road per month. In Virginia, where the convicts are furnished to the counties by the state free of cost, the cost will be about one-half of the above to the counties.

Specifications and Methods of Construction.—The top-soil where found close to the road to be surfaced can be hauled with drag or wheel scrapers and if too far away for these it is hauled with slat-bottom wagons. This method has proven economical and very satisfactory, especially where the material is close enough to be hauled with scrapers, as the scrapers do a great deal towards packing the material.

The following are the specifications of the State Highway Commission of Virginia for "top-soil" roads:

> is to be furnished by the county in beds. All stones over 3 ins. in diameter are to be thrown out. The soil is to be placed on the road, and when completed shall be 10 ins. in thickness in the center; 6 ins. in thickness, 6 ft. from the center, tapering to a feather edge 10 ft. from the center, except in cases where in the opinion of the engineer in charge of the work these thicknesses should be increased or diminished. The surfaced roadway is to be 20 ft. in width.
>
> The soil is to be kept in position on the roadway by use of a road machine, or other means satisfactory to the engineer in charge of the work, until the soil has been thoroughly wet through to the sub-grade. The soil or surfacing is then to be reshaped and dragged with a split-log drag until, in the opinion of the engineer in charge of the work, it is thoroughly dry and has set or hardened true to the cross-section called for in the specifications, or true to a cross-section which the engineer in charge of the work has specified the finished road should present.
>
> The finished road should have a crown of not less than ¾ in. to the foot and not more than 1 in. to the foot.

The road should not be considered completed the first time the surface is put in good shape, as bad sections often develop after the surface has been in shape for as long a period as one year. The real tests for a soil road are protracted droughts and long-continued, slow, drizzling rains, followed by freezing. The last mentioned condition is very trying on soil roads and the best constructed road will very probably require some attention after such weather. If the road has been properly constructed a split log drag used at the proper time will put the surface in good shape again. The proper time to use the drag is when the top is dry enough not to stick and still wet enough to pack.

Cost.—On account of the great variance in local conditions it is impossible to set one cost for soil roads. However, I will say that under the most favorable conditions a fair soil road may be built for about $400 per mile, and when built under adverse conditions may cost as much as $1,500 per mile. The seventh annual report of the State Highway Commission of Virginia shows that about 270 miles of soil and sand-clay roads were built in Virginia during the fiscal year ending Sept. 30, 1913, at an average cost of $883.32 per mile.

Advantages.—Some of the advantages of soil roads over other types of road are in cheapness in cost of construction; cheapness and simplicity of maintenance, and saving on horses and vehicles, being much easier on horses than hard surfaced roads.

With a few exceptions the roads that have been improved with sand-clay or soil in Virginia would not have been improved for many years to come and possibly never with a more expensive construction; and in sections where the sand-clay or soil roads have been built the large increase in taxable values will probably provide a sufficient fund to resurface with as good or a better material within a few years if necessary. In conclusion I would say that, from my observation through the parts of this state with which I am familiar, I think where the soil can be found convenient to the road that is to be surfaced that, considering the low cost of construction and maintenance, it is one of the most satisfactory road materials available for roads with light traffic, such as a large part of the rural roads in Virginia are called upon to carry.

Road Institutes.—The Fifth Annual Road Institute of the Northwestern Michigan Highway Officials was held March 24, at Traverse City, Mich., and on Thursday, March 26, will be held the first annual Road Institute of the Southwestern Michigan Highway Officials at Kalamazoo, Mich. The state highway commissioner, Mr. Frank H. Rogers, addresses the meetings of both institutes, his subjects being "Relation of State Highway Department to Counties and Townships," and "Repair and Maintenance."

Engineering and Contracting

Devoted to the Economics of Civil Engineering Design
and to Methods and Cost of Construction

Volume XLI. CHICAGO, ILL., APRIL 1, 1914. Number 13.

The Utility of a Map and Profile Showing the Hydraulic Elements of Water Supply Projects.

In a well prepared engineering report alternative projects should be so clearly outlined that those for whom the report is prepared can readily grasp the major features of the several alternatives as the first step in making an intelligent choice between them. The layman finds the sketch plan and profile of the hydraulic elements of water supply projects of especial helpfulness in his efforts to form a rapid and intelligent estimate of the relative worth of a series of projects. Such drawings could be incorporated to advantage in many engineering reports, in some articles contributed to technical journals, and in sets of detailed plans submitted to contractors in asking for bids on construction work.

By way of explaining more fully the character of the drawings which we have in mind the following illustration is given. In reporting to a southwestern city upon an additional water supply a consulting engineer presented the chief projects in the form here under consideration. The recommended project, like the others, was shown by means of a map and diagram which indicated, in small space, the principal hydraulic elements of the suggested works for the new water supply. The project contemplated the diversion of water from a river at a point about 30 miles from the city, and its conveyance by a gravity aqueduct to a reservoir about three miles from the city, and a development of water power which would do the pumping over half the time.

The map prepared shows the railway lines, streams and cities in the region traversed by the aqueduct. It also shows the relative positions of the head works, the diversion canal, the two sections of covered conduit at the downstream end of the diversion canal, the section of tunnel joining the two sections of covered conduit previously mentioned, the storage reservoir near the city, the conduit from this reservoir to the pumping and water power stations and of these two plants.

The profile shows the relative positions of the same elements shown on the map in their relative positions with respect to the datum plane and gives the length and grades of all conduits. It also shows cross sections of all conduits and indicates their carrying capacities, and gives the reservoir capacity. It shows, further, that the headworks consist of a low diversion dam and a silt settling basin. The elevations of the surface and bottom of the river at the point where the diversion is suggested also are shown as is the elevation of the tail race below the water power plant. The map and profile are drawn of course, to the same horizontal scale and when the two drawings are placed side by side the elements of the project, as enumerated above, stand out so clearly that the intelligent layman can readily and quickly catch the nature and scope of the works suggested. Moreover he will be able to gain a clear idea of the relation of each element to other elements and to the project as a whole.

The advantage of giving the layman, say a member of a city council, such a clear and accurate view of what is proposed by the engineer does not call for additional comment from us at this time.

Drawings of the type described seem also to have a legitimate place in the [illegible] of some articles contributed to technical journals and in the introductory paragraphs of some papers presented before engineering societies. Articles and papers are much more likely to be read if the reader can catch their "drift" with little effort through well written introductory remarks and well prepared sketch maps and profiles of the type described. Contributors to the engineering journals and to the proceedings of engineering societies will do well to bear this point in mind, especially with reference to the drawings mentioned. Of course in many articles, perhaps in the great majority of articles, there is no occasion for the presentation of such drawings, but where they are appropriate they should be used.

Perhaps others besides the editors have noted the reluctance with which contractors "wade into" a formidable set of detailed blue prints prepared to govern construction and issued as a guide to bidders and builders. The average contractor bids on many more jobs than he secures and some thought on the part of the engineer with a view to saving the contractor's time in making bids will work to the common good. To illustrate, let us suppose that bids are to be asked for the construction of the water supply works briefly described above. Many sheets of drawings would be required to show all the details of the designs. We feel sure contractors would benefit in saving of time and nervous energy if the first sheet of the set of drawings was made to show the elements of the proposed works by means of a sketch map and profile similar to the one used by the engineer in his report recommending the project to the city officials.

While our remarks have been limited to water supply projects they have equal application, at times, in other fields.

The drawings mentioned are not now commonly used for the purposes enumerated. The preparation of such drawings will add little in the way of expense and their intelligent use will effect a real saving in time, expense and energy. We consider them of especial utility in connection with sets of detailed plans, and their more frequent use in such plans will surely decrease the expense and increase the intelligence of bidding.

The Tunkhannock Creek Viaduct at Nicholson, Pa.

A striking example of the use of reinforced concrete for a high railroad structure is the Tunkhannock Creek Viaduct at Nicholson, Pa., this viaduct being a part of the extensive grade improvement work now being constructed by the Delaware, Lackawanna & Western R. R. between Clark's Summit and Hallstead, Pa. The viaduct consists of two 100-ft. and ten 180-ft. reinforced concrete arches supporting spandrel walls upon which rest a series of small concrete roadway arches. The maximum height of the viaduct from bottom of footing to top of masonry is 304 ft. We are publishing in this issue an article by the engineer in charge of construction, describing the principal design and construction features of this important structure. For a bridge of this size the layout of the contractor's plant is an important matter, this layout being illustrated and described in detail. On account of the length and height of the viaduct a 3000-ft. cableway was installed and is being used to construct the centering for the arches, and the bridge superstructure. Derricks mounted on high concrete pedestals were also used to a considerable extent on the work. The excavation for the piers was carried on inside of steel sheet piling, the depth of excavation being such that two 20-ft. sections were usually required, one section being driven and the material excavated and another then driven a short distance inside the first section. The layouts of the mixing plants are particularly effective in permitting the materials to be cheaply handled. The design of the viaduct was such that duplicate forms could be used, and the contractors have taken advantage of this to an unusual extent. The erection of the heavy steel centers of 180-ft. spans by means of the cableway is an interesting feature of the work. The general layout of the plant and the construction features of this viaduct are of particular interest to contractors, as the magnitude of the work required special equipment and ingenious construction methods.

The Development of Engineering Experiment Stations.

The University of Wisconsin is the latest to establish an engineering experiment station, it having been announced recently that such a station is to be created in the College of Engineering to take charge of the testing and research work of the college. The first engineering experiment station in this country was established by the University of Illinois in 1903. Since that time similar stations have been created by a number of engineering schools, and the movement is becoming quite general. These experiment stations have already proved their value, and engineers throughout the country are familiar with the bulletins which are being issued from time to time treating of important engineering problems. These stations have not only created a large fund of experimental data of vital interest to engineers, but they have also trained a considerable number of investigators who have demonstrated their ability to conduct tests and to analyze the results in an efficient and impartial manner. Anyone not familiar with this kind of work can hardly appreciate the high degree of patience, skill and inventiveness which must be possessed by the successful investigator. There is nothing more confusing and misleading than a mass of experimental data which has been obtained by untrained investigators. We all know that such data—like expert witnesses—can seemingly be made to prove almost anything which those in charge may seek to demonstrate. As these experiment stations become more prevalent, and as they enlarge the scope of their activities and build up efficient corps of trained investigators, their influence will undoubtedly be one of the strongest factors in the development and recognition of the engineering profession.

Difficult Cofferdam Construction in the Mississippi River at St. Louis.

Since 1892 St. Louis has drawn its entire water supply through one intake tower and tunnel. Appreciating the extreme hazard for a city of such size to be dependent upon a single intake the present water commissioner prepared plans for a new intake tower, tunnel and screen chamber and contracts for the construction of these works were awarded in June, 1913. The design of these works was the subject of an illustrated article published in this journal for October 29, 1913. The construction is now well advanced and in this issue we publish an article describing and illustrating the construction methods and plant employed in placing the cofferdam for the new intake tower.

The difficulties encountered in unwatering the site for the new tower will be readily appreciated when the conditions are understood. The river is from 18 to 22 ft. deep at that point and the current ranges from 3½ to 4 miles per hour. Moreover, the river is subject to sudden and great rises. Troubles from ice jams also had to be anticipated.

The article to which we refer describes the design and construction of the floating plant built to float the timber cofferdam caisson to place, the construction of the caisson, the methods used in anchoring the barges and caisson in place, the launching of the caisson and also describes the concreting of the footings of the intake tower.

The article was prepared for this journal by the general superintendent of construction operations. The river portion of the intake tunnel is over 2,000 ft. long and 8 ft. in clear diameter when concrete lined. Rapid progress has been made in driving this tunnel. The plant and methods employed in the tunneling operations will form the subject of another contribution now being prepared for early publication in the water works section of this journal.

Is Upward Hydrostatic Pressure or Low Coefficient of Friction Responsible for Recent Dam Failures?

Every now and then a dam founded on rock slides out. And as often as such a failure occurs it is asserted that the cause was probably upward water pressure upon the bottom of the dam. We have taken occasion more than once to express doubt as to the adequacy of such an explanation. We now wish to outline what should be done to *prove* the cause of recent dam failures.

It seems clear that one of two causes, or possibly a combination of the two, is responsible for many failures of masonry dams founded on rock: (1) Upward hydrostatic pressure on the base of the dam; (2) Low coefficient of sliding friction.

The theory of upward hydrostatic pressure assumes that the pressure increases uniformly from zero at the down stream toe of the dam to a pressure due to the full head of water at the upstream heel of the dam. But, upon what tests is this theory based? If the rock under the dam contains vertical fissures and is otherwise impervious, theory would indicate that the upward pressure would not exceed that due to the cross-section area of the vertical fissures. If there are also horizontal fissures beneath the dam, then the cross-section area of these also must be taken into consideration. Let us assume, however, that there are no fissures worthy of consideration, but that the rock beneath the dam is very porous, and that water flows steadily through these pores or voids. This is evidently the tacit assumption underlying the theory that the upward pressure increases uniformly from zero at the toe to full pressure due to the head at the heel of the dam. Is it not clear that if this is the case the *total* area of dam base is not under pressure? Are not the voids in the rock so many small pipes, and is not the combined cross-section area of these pipes far less than the area of the base of the dam? An affirmative answer seems imperative, but once this is conceded the upward pressure theory loses most of its weight.

This, however, is not the only defect in the upward pressure theory. When water comes in contact with a solid, it adheres to the solid by virtue of molecular attraction between the solid and the water. In voids of capillary size this molecular force becomes of exceeding importance. It is this force, for example, that makes wet sand on a sea beach adhere so that it forms a good road, although the same sand when dry falls apart and will not support a wheeled vehicle without much displacement. What is the effect of the molecular attraction between water and the walls of capillary tubes through which it flows? Can anyone affirm that it does not greatly reduce the upward pressure on a dam founded on rock containing pores through which water flows? We know of not a single experiment that can be appealed to in answering these questions.

Here, it seems to us, is an excellent theme for an engineering student's thesis—indeed for many a thesis. At least some of the fundamental principles can be solved by quite simple experiments on a small scale. As matters now stand, engineers do not know even the fundamental principles of hydrostatic pressure of water in porous solids underlying dams.

There is an equally fruitful field for experiment on the coefficient of sliding friction of rock and earthy materials. Experimental data on this subject are lamentably inadequate, and such data as we have are based on small unit pressures. It is well known that coefficients of friction are a function of unit pressure. Hence coefficients obtained from tests under small unit pressures cannot be safely applied in cases involving large unit pressures. Yet this is precisely what engineers who design dams are doing and have done. Their factors of safety have been adequate in most instances, to protect them against erroneous coefficients of friction.

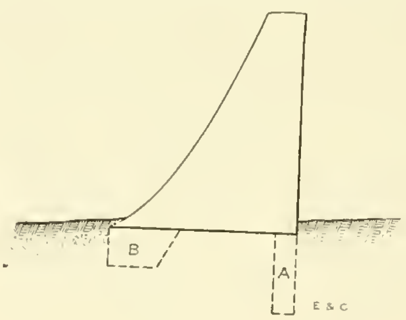


Fig. 1.

We have reason to believe that the coefficient of friction of very hard rock on its kind is greater the greater the unit pressure. But for soft rock (shale, etc.) we believe the converse may be true. We have little doubt that the coefficient of friction of clay decreases rapidly with increase unit pressure, particularly when the clay is wet. Tests should be made on a great variety of rocks and earths, dry, damp and very wet, under varying pressures up to 200 lbs. per square inch. It should be remembered that the overturning moment throws a greater pressure on the toe of a dam than that due to the weight of the superincumbent masonry. Hence the desirability of carrying the friction tests up to at least 200 lbs. per square inch pressure.

Slight jarring reduces the apparent coefficient of friction where the unit pressure is slight. Is it not likely that heavy jarring may have the same effect where unit pressures are great? If so, is it not evident that when a dam is trembling as a result of the water falling over its face, the dam may be moved by sliding more easily than when it is quiescent?

It is well known that even the slightest sliding is accompanied by a great decrease in the coefficient of friction. Hence the importance of preventing incipient sliding, entirely aside from any question as to the effect of fissuring.

A masonry dam may be perfectly safe against sliding on its base, yet the sub-base may cause failure by sliding on itself. This last is what probably happens in most cases of failure by sliding. Let there be a horizontal layer of soft shale, clay or the like, even several feet below the base of the dam, and there will be grave danger of a slipping along this plane of low frictional resistance. Hence the importance of core drilling where dams are to be founded on stratified rock, particularly where the strata are horizontal or dip down stream. The cores will show whether excavation should be carried deeper or whether a heavier dam should be designed.

Whether upward pressure or low coefficient of friction has caused dam failures is a matter of extreme importance. Those who adhere to the upward pressure theory would be satisfied with a core wall at the heel of the dam, to cut off percolation. But an engineer who regards low coefficients of friction as the cause of frequent failures would place less confidence in core walls built in rock. He would reason that capillary action would eventually result in wetting the rock sufficiently to reduce its sliding friction, in spite of the best core wall possible. He would, therefore, either increase the weight of the dam or excavate the foundation deep enough to secure a fairly deep layer of rock in front of the toe; for in the latter case he would oppose sliding not only by the weight of the dam but by the weight of a layer of natural rock below the dam.

Fig. 1 shows the difference in design that would follow the adoption of the two theories. A believer in the upward pressure theory would build the deep core wall, A, at the heel of the dam. A believer in the low friction theory would build the relatively shallow but wide wall, B, at the toe. The wall, A, would be designed merely to prevent water from flowing under the dam. The wall, B, would be designed to get a "toe hold" on the rock ledge below the dam.

This illustration will serve to indicate that the two theories must lead to radical differences in the design of masonry dams. It is highly important that tests be conducted, as above outlined, to establish to what extent each of the theories is tenable.

Water Works

Construction Plant and Methods Employed in Placing the Cofferdam for the New Intake Tower of the St. Louis Water Works.

Contributed by C. H. Hollingsworth, General Superintendent of Construction.

There is now under construction at St. Louis a new water works intake tower located in the Mississippi River, near the Illinois shore, and a 2,789-ft tunnel leading from the tower to a screen chamber at the present pumping station at the Chain of Rocks on the Missouri shore.

The contract for the screen chamber, tunnel and intake tower was awarded in the spring of 1913 and work was started about July 20. As this article treats only of the unwatering of the site of the intake tower it will only be said in passing that the tunnel is now (March 12) about completed.

The difficulties of unwatering the site for the intake tower were great as it is located in from 18 to 22 ft. of water, at normal river stages, on a rock bottom which is very uneven, and in a current of 3½ to 4 miles per hour. Added to this is the fact that the river is subject to sudden and heavy rises and, in the winter time, is filled with floating ice which frequently and suddenly jams at various points and as suddenly lets go again.

The various methods of unwatering cofferdams were considered, the most of them were discarded as unsuitable. Finally the method of constructing an annular caisson large enough to enclose the entire tower with several feet to spare was adopted.

CONSTRUCTION AND TRANSPORTING OF CAISSON AND BARGES.

The caisson is built of squared yellow pine timber, contains, altogether, 200,000 ft. B. M. of timber. It is 94 ft. long and 52 ft. wide over all, giving a space 78x36 ft. inside. This llows about 4 ft. space on all sides between he tower and the cofferdam. The caisson was artly built on four barges and was towed bout 8 miles up the river from St. Louis to he site of the tower and was there completed.

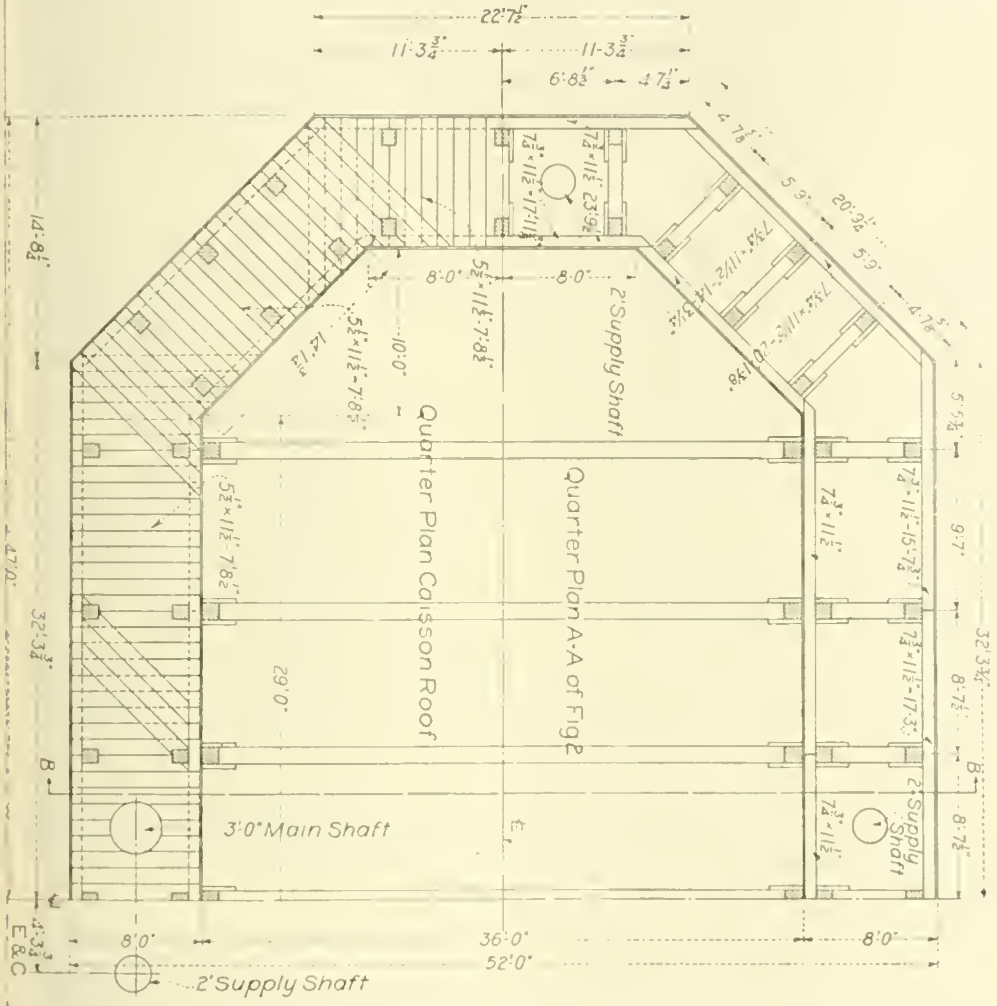


Fig. 1. Quarter Plans of Caisson, Employed in Constructing New Intake Tower of St. Louis Water Works, at Roof and Below Deck.

The caisson is shown in part plan in Fig. 1 and n vertical section in Fig. 2.

Construction work was commenced July 26, 1913, on the barges for floating the caisson and they were launched at the rate of one a week thereafter until the four had been completed. These barges were 48x20 ft. in plan and 5 ft. 2 ins. deep. The bow was built with a rake end or overhang of 6 ft. and the stern was square. They had one bulkhead running fore and aft in the centre. The gunwales were of 4 in. x 12 in. planks outgaged to form a caulking seam. The bulkhead was also of 4 in. x 12 in. plank without the caulking seam. The bulkhead was in the center of the barge and halfway between it and the sides were two streamer planks thoroughly braced and tied to the cross frames. The decks were of 2 in. planks outgaged and caulked. Altogether this construction made very stiff barges and strength was requisite as they were required to withstand hard usage. Vertical sections through the barges are shown in Fig. 3 and detail of end and cross frames in Fig. 4.

When all four barges were in the water, launchways of 8 in. x 12 in. timber were laid athwartships on their decks. A sketch of the launchways is shown in Fig. 5a. These timbers were laid flatwise and reached entirely across the barges. The tops of the launchways were greased with tallow and butterboards of 4 in. x 12 in. planks with lugs on them to keep them from slipping laterally were laid on top of the launchways. Then the barges were spaced the proper distance apart and held in position by 12 in. x 12 in. timbers bolted to the timber heads and stiffened by diagonal braces of 2 in. plank spiked to the decks of the barges and to the tie timbers.

The cutting edge of the caisson was laid down on the butterboards and both walls of it were carried up to the deck. The deck was then placed and the walls were carried up to four courses above it, before the caisson was towed to its final location. Figure 6 shows the spacing of barges and the location of the caisson on same and the method of bracing barges apart.

The four barges with the partly finished caisson on them drew about 30 ins. of water and were handled by one tow boat. The distance of about 8 miles was covered in 2 hours and 15 minutes, the boat starting from North Market St., St. Louis, at 10:30 a. m., and arriving at its location at 12:45 p. m. It was made fast to the anchor cables at 2:30 p. m.

The problem of anchoring the caisson was rather complicated in the swift current. Upstream the bottom was rock with pockets of sand; on the west side it was the same and on the east side it was loose river sand. The best way to provide anchorage was to drill holes in the rock of the river bottom and let in ring bolts for the upstream and the west anchors and to use concrete blocks for the east anchor.

METHOD OF SETTING ANCHORAGES FOR FLOATING PLANT IN STREAM BOTTOM.

It was entirely too expensive to equip a boat with spuds in order to hold it steady to drill from and it was also out of the question to drill from a raft or floating boat as the current and wind would swing it from side to side in spite of anchors, while waves from passing boats would cause it to rise and fall from 2 to 3 ft. Therefore a drill platfrom to rest on the river bottom was constructed. The details of this platform are shown in Fig. 7 and a view of the platform in place with the drill mounted and ready for use is shown in Fig. 8. It was composed of a concrete block with a section of 12-in. pipe imbedded in it and a timber platform fastened to the top of the pipe. The concrete block was 6 ft. square and 3 ft. thick, the pipe being imbedded in it for 2 ft. The pipe was 22 ft. long and as the bottom of it was 1 ft. above the bottom of the block this brought the top of the pipe 23 ft. above the river bottom when it rested on the bottom. It was braced to the concrete block by means of eight 1-in. reinforcing bars, the lower ends of which were cast in the block near its outer edges and the upper ends of which were clamped to the pipe by a heavy collar at a point about 4 ft. above the block. On top of the pipe was a standard flange and below it were two 6 in. x 8 in. timbers 6 ft. long running parallel to each other and at opposite sides of the pipe. These timbers were bolted together and bolts running down through the flange kept them from slipping down. Across the top of the 6x8's and spiked to them were laid 3-in. planks 6 ft. long the whole forming a platform 6 ft. square. Holes were left through the platform alongside the pipe to allow a chain to be passed around the pipe and up through platform to admit of the whole being lifted with a derrick. From the pipe directly under the platform a ladder ran down to the concrete block and was fastened securely to both pipe and block.

When this platform had been assembled it was handled by a tower derrick mounted on a 30 ft. x 60 ft. barge. A view of the derrick barge handling the drill platform is shown in Fig. 9. This derrick was intended for use in setting stone, steel, etc., on the intake tower itself. The timber tower was 30 ft. square and was set 32 ft. above the deck of the barge. It

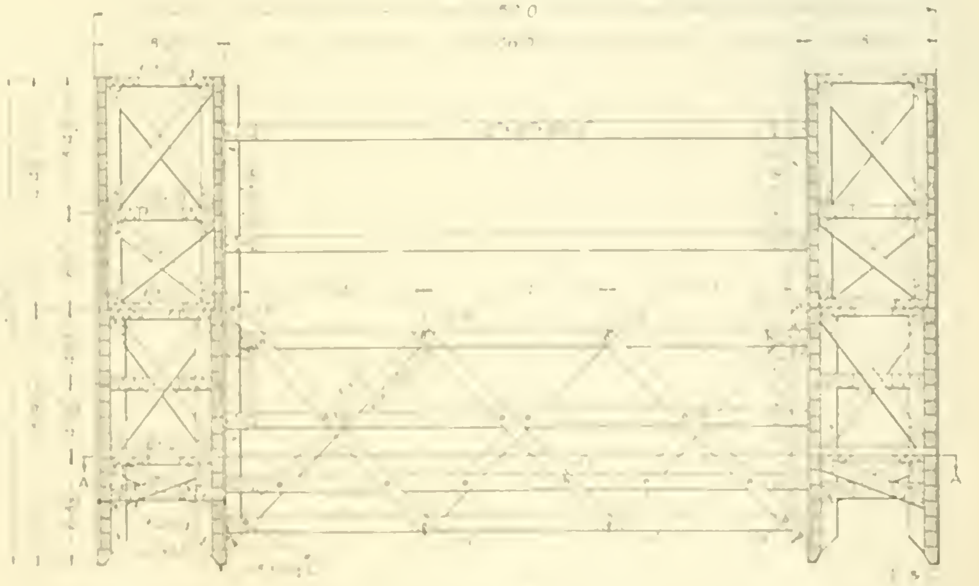


Fig. 2. Vertical Section B-B of Fig. 1, Caisson for St. Louis Intake Tower.

was built of 12x12in. and 12x16 in. timbers and strongly braced. On top of this tower a combined A-frame and stiff leg derrick was erected with a 32-ft. mast and a 50-ft. boom. As the tower and derrick were set back about 12 ft. from the bow of the barge there was plenty of room in which to place the drill platform when not in use or when changing from one hole to another. The holes for the upstream anchors were drilled first and to drill them the derrick boat was anchored with the stern up stream by means of two lines running to two concrete anchor blocks previously placed. The barge was then directed into position north and south by the engineers from a base line previously established and set on position east and west by means of an instrument and a stadia rod. The barge was moved east or west until the rod reading checked with the distance which had been worked out by triangulation. Figure 10 shows a plan of the derrick barge and the method of anchoring it and the drill platform while drilling holes for anchor bolts.

The drill platform was lifted by the derrick and lowered until the concrete block rested on the river bottom. In spite of the weight of the block and platform (about 16,000 lbs.) the current would sweep it down stream a considerable distance and spin it around. To prevent this lines had to be made fast to the foot block to keep it from drifting and other lines were made fast to the ends of 2x4-in. pieces spiked along the sides of the platform to prevent it from spinning. When the concrete block rested firmly on the river bottom and the derrick had been unhooked, the drill, an Ingersoll D-24 machine, was set up on the platform and the drilling started. Often it would require several attempts to get the block resting firmly and evenly on the bottom as the latter was very irregular. In addition to this difficulty there would sometimes be considerable sand under the block which would have to be removed before the drilling could be commenced. In such cases a diver would go down by means of the ladder from the platform taking the end of a water or steam hose. With this the sand would be washed or blown out from under the block, thus allowing it to settle firmly. Then in order to keep the silt from difting under the block the upstream side was protected with packed sand bags. The hole in each case was started with a 24-ft. steel and when this had been run out it was followed by a 26-ft. steel. The first steel measured 3½ ins. across the bit while the second measured 3¼ ins. The length of the bolt to be inserted in the hole was then carefully measured and the hole was drilled accordingly. The depth averaged about 4 ft. and it was about 3¼ ins. in diameter at the bottom. After the drilling of the hole the ring bolt was lowered down carefully through the 12-in. pipe by means of a ½-in. cable attached to the ring. These bolts were split at the bottom and a wedge was inserted in this slot. A detail of the 3-in. anchor bolts is shown in Fig. 5b. The drill hole would break out around the top leaving a cone shaped hole about 8 ins. across the top and tapering to the regular diameter of the hole at about 10 ins. below the surface of the rock. This made the entering of the bolt in the hole a comparatively easy matter except that it had to be done very quickly or else the fine sand carried by the river water would fill the hole in a few minutes. After losing one hole that way, however, matters were arranged so that no time was lost in getting the bolt into the hole as soon as possible after the hole had been drilled. The drill was hoisted off the platform by the derrick which also placed the ring bolt. A piece of steel shafting 6 ins. in diameter and 12 ft. long was swung on a single or runner line from the derrick boom and after care had been taken that the ring was swung over to one side of the pipe the ram was lowered until it rested on top of the eye. It was then lifted a few feet and dropped repeatedly, driving the bolt down firmly to the bottom of the hole and forcing the wedge up into the slot and expanding the sides of the bolt tightly against the sides of the hole. In this way all bolts were set down firmly into the holes with their eyes hard against the rock of the river bottom. When the bolt had been driven home the derrick was hooked to the platform and the latter raised high enough to clear the ring of the bolt and the lines leading to the foot block were slacked enough to allow the current to drift the platform down stream a few feet when it was lowered again clear of the ring bolt and down stream from it. The diver then went down the ladder which was always kept on the upstream side of the platform and found the bolt by the simple method of getting hold of the ½-in. cable leading down through the pipe to the ring and following it to the ring. The end of this cable had previously been made fast on the barge and

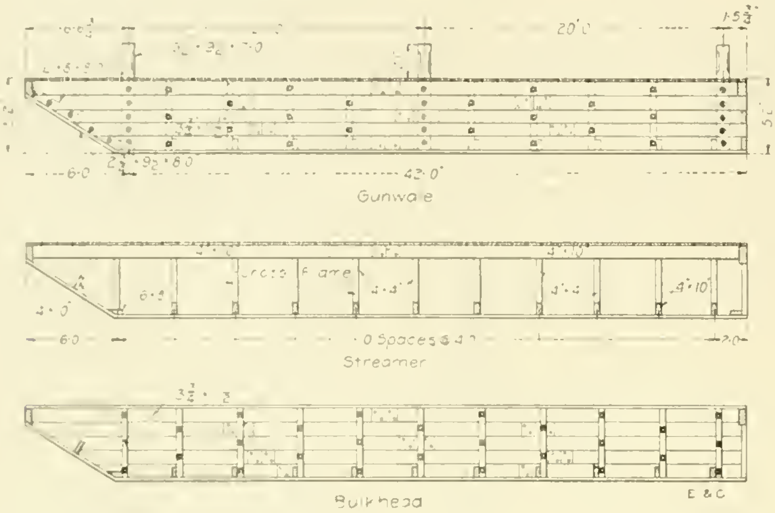


Fig. 3. Vertical Sections Through Gunwales, Streamers and Bulkheads of Barges for Floating Cofferdam Caisson Into Place, St. Louis Intake Tower Construction.

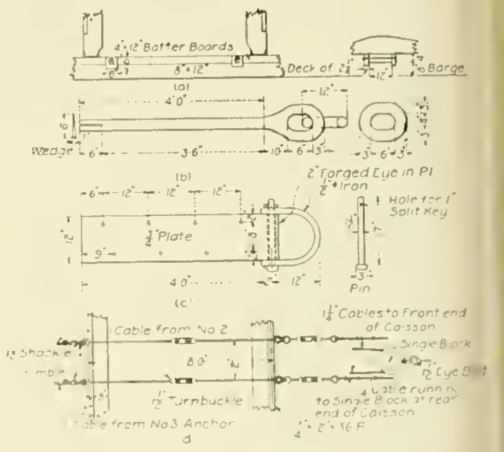


Fig. 5. Details of Launching Ways and Anchorages for Cofferdam Caisson, St. Louis Water Intake Tower Construction.

(a) Sketch of launchways for caisson.
(b) Details of 3-in. anchor bolts for upstream caisson anchors.
(c) Details of plate and shackle for fastening up-stream anchor cables to side of caisson.
(d) Method of attaching anchors 2 and 3 to upper end of caisson and tying through to lower end.

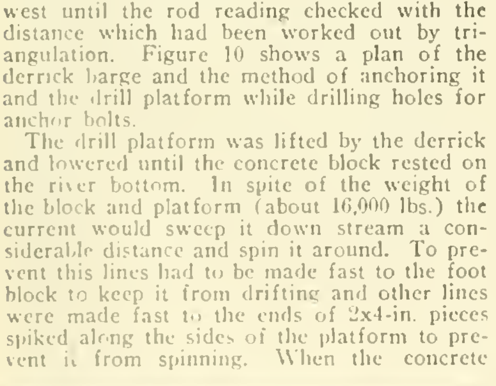


Fig. 4. Detail of End and Cross Frame of Barges Shown in Fig. 3.

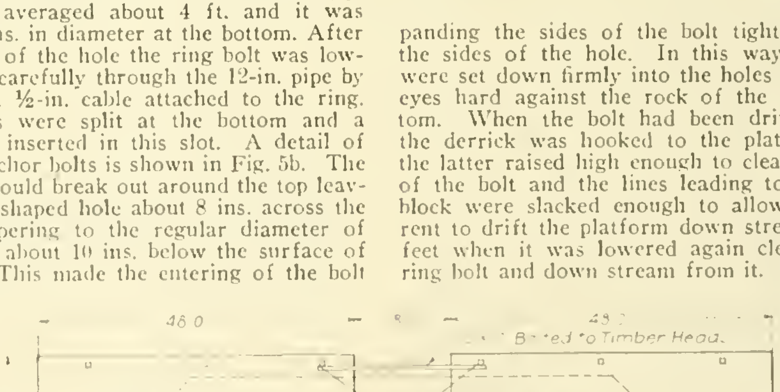


Fig. 6. Plan Showing Spacing of Barge and Location of Caisson on Same—Also Method of Bracing Barges Apart.

was left attached to the ring bolt for use as a buoy line. When the diver had located the ring bolt the end of the anchor cable was sent down to him by a hand line. This cable was attached to the anchor bolt by means of a 1½-in. shackle for the upstream anchors and a 1-in. shackle for the side anchors. A detail of the plate and shackle for fastening the upstream anchor cables to the side of the caisson is shown in Fig. 5c. When the cable had been shackled on the drill platform was lifted clear of the water, the bight of the ½-in. buoy cable caught below the bottom of the pipe, the end was allowed to run through, a buoy was made fast to it and thrown overboard and the anchor cable, which in the case of the upstream anchors was 1-in. steel cable, was coiled down on the deck of the barge.

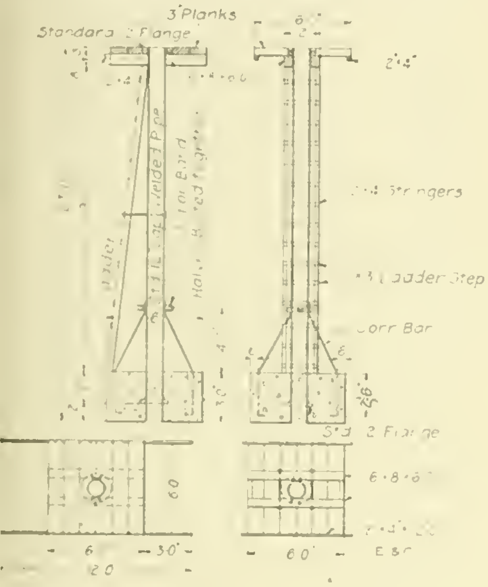


Fig. 7. Details of Portable Platform for Drilling Holes for Anchor Bolts on River Bottom.

When the four upstream anchors had been set the caisson was towed up to position. As the steamboat pushed the caisson ahead of it, it was an easy matter to work it carefully up to the downstream end of the derrick barge, take the four anchor cables aboard, make them fast and drop back until the cables were taut. After the caisson had been anchored from these cables the side anchors were placed. The west side anchor bolts were placed as already described except that the ring bolts were 2-in. instead of 3-in. Then the anchor cables, which in this case were composed of two parts of ⅝-in. steel cable for each anchor, were taken aboard the caisson and made fast.

The east side anchors as previously stated, consisted of three concrete blocks for each anchor. These blocks were 6 ft. square and 2 ft. thick. In the tops of the blocks were two loops of 1-in. round iron for use in handling the blocks and in the side of each one was a loop of 1½-in. iron for attaching the anchor cables. The bottoms of the blocks were studded with pieces of 1½-in. square iron 18 ins. long. About 12 ins. of the length was imbedded in the concrete and the remaining 6 ins. projected so that they would catch in the inequalities of the rock in case the block settled down in the sand low enough for them to reach the former. A loop consisting of four parts of ⅝-in. steel cable and about 4 ft. long was passed through the 1½-in. iron loop in each block and a piece of 1 in. steel cable about 60 ft. long run through these short cables and clipped in a loop 30 ft. long. This was to allow of flexibility enough to enable each block to bear part of the strain and to allow them to settle to a solid bearing without in any way interfering with each other. The 1 in. cable loop was long enough so that when pulled straight upwards by a derrick barge the clips on it would be above water and so allow their removal. From each set of blocks the anchor cables were run to the caisson and made fast.

The three blocks for one anchor were placed simultaneously by setting them on edge along the front edge of the derrick barge with the bottoms facing outward and with an 8-in. x 8-in. timber held up against the 1-in. loops in their tops by the main fall of the derrick on which a light strain was kept. The steamboat pushed the barge up stream until the right location had been reached when a heavy strain was taken on the main fall and the three blocks were all dumped overboard at the same time, after which the cables were taken aboard the caisson. The position of the anchors and the method of rigging up the barges to launch the caisson also is shown in Fig. 11.

A system of fastening the anchors and especially the upstream ones to the caisson had to be carefully worked out as the entire strain could not be taken by the outer wall of light timber. All anchors were fastened at a point about 3 ft. above the top of the deck and all in the same horizontal plane. In order to transmit the pull from the upstream anchors and have part of it taken by the downstream end of the structure the arrangement illustrated in Fig. 5d was made. In the upstream end, two 1½-in. bolts with an eye on each end and a turnbuckle in the center were run through both walls at this end of the caisson and 2 ft. apart, being also parallel with the center line of the crib. On the inner side of the caisson the shoulders of these eye bolts bore against a ¾-in. plate 12 ins. x 36 ins. and to give them additional flexibility 1½-in. turnbuckles were shackled to their outer ends and to the latter were shackled the 1-in. anchor cables with 1½-in. shackles. Through the two walls of the downstream end and on the center line of the caisson ran a 1½-in. eye bolt provided with a nut, jam nut, and 12-in. plate washer on the outer or downstream end. To the eye on the inside end of this bolt was hooked a steel single block through which ran a 1¼-in. cable. On each end of this cable were attached 1½-in. turnbuckles which in turn hooked into the eyes in the downstream ends of the eye bolts to which were made fast the cables from anchors 2 and 3. When these arrangements had been made the turnbuckles were adjusted so as to give the proper bearing all around. The upstream anchors 1 and 4 were made fast by means of shackles and 1½-in. turnbuckles to a heavy plate, and D-bolt at the upper corners of the caisson. The anchors from the sides were made fast by means of 1-in. shackles and 1-in. turnbuckles to eye bolts running through both walls of the caisson near the upper corners.

LAUNCHING THE CAISSON.

When the caisson had been built up to a point about 16 ft. above the cutting edge, sheathed and caulked, preparations were made to launch it. The timber heads coming inside the caisson were cut off flush with the decks of the barges and all bracing between barges was removed. In each of the four barges a 2 in. stop cock was placed about 2 ft. below the deck and in the sides which were turned inward. A 2-in. hole was also bored in the bottom of each of the barges directly under the hatch and plugged. A derrick barge was moored at each side of the caisson and about 75 ft. distant from it. They were held upstream by a ⅝-in. steel cable made fast to one of the upstream caisson anchors and one part of the ⅝-in. cable from each of the side anchors on that side was carried aboard the barge and made fast. Then a set of double blocks rove with 1½-in. manila line was hooked to each of the barges under that side of the caisson and made fast aboard the derrick barge while the hauling part of this was led through snatch blocks to the hoisting engine.

When both sides were made ready in this way the lines were pulled tight on all four barges and a slight strain put on each. The stop cocks on the barges were then opened and the plugs taken from the bottom holes allowing them to sink slowly. All four barges settled at a very even rate until the decks were awash when the upstream ones settled more rapidly than the others owing to the current carrying them under. When the decks of all four barges were under water as much strain was taken on the hauling lines as the manilla rope would stand. The filling of the barges began at 4:30 p. m., Oct. 2, and at 7 p. m. the decks were under water. A few minutes after that the buoyancy of the timber in the caisson began to lighten the load on the barges and about 7:15 p. m. three of them slid out and bobbed up to the surface like corks. The fourth barge which was the downstream one on the east side, slid out part way and then stuck. At that corner the rock was very high in the bottom and the caisson when floating came within about 3 ft. of the bottom. As the barge was 5 ft. 2 ins. high it became jammed between the crib and the bottom when all but one corner had slid free. The other barges were all caught by the steamboat and taken ashore to be pumped out. The next morning after the slack had all been taken out of the anchor cables opposite to the refractory barge, the latter was easily pumped free and the caisson floated, drawing about 14 ft. of water which was slightly more than calculated upon. Some parts of it were not caulked up to the water surface and it was rolled first to one side and then the other and the caulking carried practically to the top before any additional timber was added. This was accomplished by placing a gin pole in each corner on the same side with their bottoms resting on the river bottom and a set of blocks leading from their tops to the top of the crib. Lines from these blocks were then led to a hoisting engine and a strain taken. This, aided by the buoyancy of the crib itself, elevated that side of the crib from 3 to 4 ft. allowing the caulking to be done, after which that side was allowed to settle down and the gin poles shifted to the other side and it was handled the same way. The construction on the caisson was then resumed and the timber carried up to about 26 ft. above the cutting edge at which time it rested firmly on bottom all around.

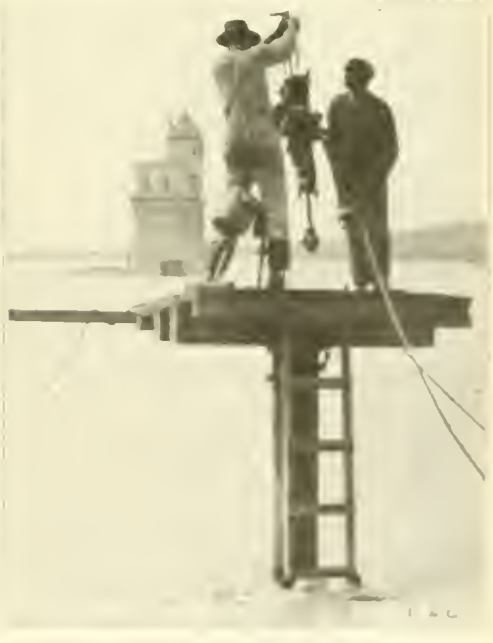


Fig. 8. View of Portable Drill Platform with Drill Mounted and Ready for Operation.

The rock bottom, as before stated, was extremely uneven and the southeast corner was 5 ft. higher than the northeast corner giving the caisson a very bad twist. Also there were depressions in the bottom where the cutting edge would be from 2 to 3 ft. clear of the bottom. When the caisson had been completed up to a point 26 ft. above the cutting edge or to about elevation 59 an attempt was made to put air on it but as the only compressor on hand was one of 675 cu. ft. per minute capacity and as the twisting of the caisson had caused the seams to open up, no headway was made as the air leaked out about as fast as the compressor could pump it in. Clay filling was then dumped in the space between the walls, on top of the deck which was a departure from the original scheme which was to fill this space with sand as ballast. The clay, however, acted as ballast and in addition helped materially to stop the leaks.

When about 10 ft. of clay had been dumped in another attempt was made to put the caisson under air. While 2 or 3 lbs. of air would be registered on the gage it was still impossible to get in through the man lock which unfortunately happened to be located on the low side of the caisson. When the air had reached about 2 lbs. pressure it had forced the water down so low in the working chamber that numerous leaks were exposed and at the same time the water was not low enough to admit men through the man lock at the low side. It was decided to cut through the deck at the highest corner and place a new man lock there. In order to do this the pocket at the high point was cleaned out and pumped dry and a section of main shaft set down on the deck. Concrete was then filled in around it and allowed to set. A lock door had been placed in position at the top of the shaft section and when the concrete had sufficiently set, air from a small compressor was introduced through the 1-in. blowoff valve in the door, after two men had gone inside. The air pressure was kept at about 6 lbs. while the men inside fastened the shaft section down to the deck with 1-in. lag screws running through the flange on the bottom of it through which were 24 holes. Then a small section of the deck was cut out to allow a man to pass inside. The air was then taken off this shaft section and a second section bolted to the top of it to form a man lock.

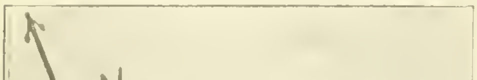


Fig. 9. View of Derrick Barge Handling Drill Platform—Platform Shown Suspended Ready to Lower to River Bottom.

Meanwhile it was thought desirable by grouting to tighten up the deck. Two inch pipes were pushed down through the clay until they brought up against the top of the deck. These pipes were pushed down alongside of each vertical timber and one each side of same. They were then cleaned out by means of a blowpipe of ¾-in. pipe using high pressure air, after which they were grouted with about 60 lbs. of air using neat cement. A Ransome-Canniff grout machine was used for this. It was interesting to note that although the grout pipes had nothing to hold them but their friction against the clay, they never in any case blew out or even moved when the pressure was applied with over 60 lbs. of air on the grout machine. Also, it was rarely that any of the grout came up alongside the pipes and when it did so it was only after all other crevices had been filled and the pipe had been blocked. Often though the holes were grouted to a standstill and the grout pipe completely blocked with the 60 lbs. pressure none of the grout would appear on the surface.

After the grout had set air was again applied and the pressure run up to about 4 lbs. Men then went inside through the lock which had been cut in on the high side of the caisson and clayed up all seams thus stopping the leaks and allowing the air pressure in the working chamber to be raised to about 6 lbs. at which point the cutting edge on the high side was exposed. The rock bottom along the upper side was cleaned of sand and gravel and the outside and inside cutting edges were packed with clay sacks. This side was then concreted to a depth varying from 18 ins. to 5 ft. according to the bottom. In all cases the concrete was carried up to a point 2 ft. above the cutting edge. When the upper side had been sealed the air pressure was raised until the cutting edge on the lower side had been exposed when it was packed with clay sacks and concreted also.

One disadvantage in the concreting operations was that it was impossible to expose the rock at all points for in order to do that the air pressure would have had to be raised so high that it would have blown out under the cutting edge as much of the bottom was from 1 to 5 ft. below the latter. As the water was muddy the state of the bottom had to be determined by feeling with an iron bar, to see whether or not it had been sufficiently cleaned. When the concreting was going on the pumps were being set up to pump out inside the crib. These included a No. 6 direct connected centrifugal pump with a capacity of about 1,000 gals. per minute, one belt driven No. 5 centrifugal pump with a capacity of about 800 gals. per minute and two 6-in. steam siphons with a combined capacity of about 2,500 gals. per minute.

UNWATERING OPERATIONS.

When the caisson had been sealed down to the rock air was still kept on it and the pumps were started. They lowered the water level inside the crib to a point about 5 ft. below the river level after which it came back up, to its former level, showing that as soon as any head was put on from the outside a large leak had started. A sand dredge with a 15-in. centrifugal pump on it was obtained temporarily to see if the water could not be pumped down so we could get at the leak. This pump was rated at about 6,000 gals. per minute capacity. The dredge was tied up alongside the caisson and the suction dropped inside the latter. When this was in position it and all the other pumps were started and again the water was pumped down about 7 ft. and then it started to rise again. The pumps were kept going and a diver sent down to find out just where the leaks were.

His examination showed that the greater part of the leakage was from the west or low side of the caisson.

The pumps were again stopped and men were sent inside the working chamber to cut down through the concrete at the points where the water came through under it. This was a slow job as there was very little room to work and the concrete had set very hard. At first a hole about 2 ft. square was cut through the concrete in each of the pockets along the west side and when it was found that there was a cavity under the concrete these holes were connected by a slot about 18 ins. wide. Numerous cavities were found with boulders in them and at other places quantities of loose gravel would be found. In the former case there had been hard gravel in the holes with boulders wedged tightly together on top and to anyone sounding the bottom with an iron bar the boulders would appear the same as the solid rock. When the pressure came from the outside, however, the small gravel and sand washed out leaving the boulders in the hole. Where the loose gravel was found, it had once been concrete but when the west side had been partly concreted someone with a candle who was looking for leaks had set the oakum in the seams on fire and in order to put that fire out the working chamber was flooded. To do this the air was shut off from the compressor and the air in the caisson was allowed to escape. As the bottom layer of concrete had been placed all around at that time the only way for the water to come into the working chamber was for it to come up through the green concrete on the west side. This washed the cement out. When the fire was out a part of this concrete was removed and the remainder was remixed using additional cement. Some of it had been overlooked, however, hence the trouble.

When the loose gravel and boulders had been cleaned out the holes under the concrete were filled with a rich mixture of concrete except directly under the slot that had been cut along the center of the working chamber. From the rock up to the top of the concrete this slot was filled with neat cement making a cut off wall and the cement set up quickly. Where it was found that the concrete had been bonded firmly to the rock upon cutting the 2-ft. hole first referred to nothing was done except to refill the hole with neat cement mortar. When this had set, the pumps were again started but meantime the entire pumping plant had been changed. The 15-in. sand dredge had been taken away, and as the two 6-in. syphons used a great amount of steam for the amount of water pumped and as the rest of the plant was entirely inadequate to cope with the situation a direct connected 12-in. Morris centrifugal pump was obtained with a capacity of 4,200 gals. per minute. In addition a 5-in. Emerson and an 8-in. Emerson pump were installed having a combined capacity of about 4,500 gals. per minute.

In starting to pump out, however, only the 12-in. and the 5-in. centrifugal pumps were used and these pumped the water down so that the bottom was exposed in about two hours. The bottom was first cleaned of silt of which there was a deposit about 3 ft. deep and then the sand and gravel was excavated. There seemed to be some very extensive leaks after the sand and gravel had been removed as the pumps continued to handle about 2,500 to 3,500 gals. per minute according to the stage of the water in the river. The leakage came in chiefly in the northwest corner and midway along the west side. At the latter point it was known positively that the concrete was sealed tightly to the rock but in the northwest corner the first examination had shown that there the concrete was on rock bottom and a hole had been cut in several pockets with no slot between. From the leakage which showed it was thought that there might be a crevice between two of the holes. As the water came in so heavily there was very little chance to make an examination inside the cofferdam and men were again sent inside the working chamber and a slot cut

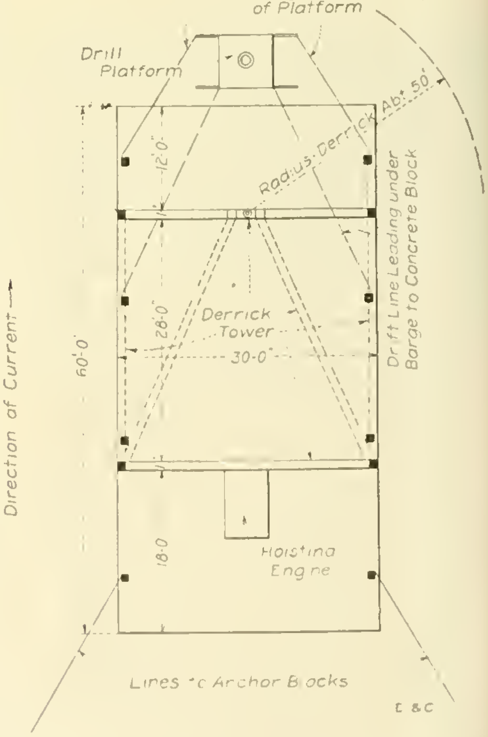


Fig. 10. Plan Showing Method of Anchoring Derrick Barge and Drill Platform While Drilling Holes for Anchor Bolts.

in the concrete for a distance of about 15 ft. and down to the rock. This showed that the concrete was firmly bonded to the rock at all points and that there could be no leakage between the two.

At the same time all loose material was being removed from inside the cofferdam as rapidly as possible and especially near the corner where the worst leak seemed to be. This leak when uncovered proved to be a horizontal seam which came to the surface about 10 ft. inside the cofferdam and dipped rapidly so that it must have been about 4 ft. below the surface under the center of the working chamber. From this seam a vertical seam led downward about 10 ft. being about 6 to 8 ins. wide. These seams could not be caulked so a bulkhead of 2-in. plank and timbers was built across the corner, grout pipes were inserted in the seams with the tops of same above water level and then the space inside the bulkhead was filled with clay and grout was forced into the pipes stopping the greater part of the leakage from that source.

As the bottom rock was cleaned off, however, and the surface rock pried loose other leaks developed and incidentally the cause of the difficulty in getting the caisson sealed tightly to rock became apparent. In the rock which was a fine grained hard limestone, there were holes and watercourses worn which were tightly packed with gravel and sand and with small flat boulders wedged tightly on top. These latter could not in some cases be removed with a pick and had to be pried loose with a bar. In addition to these there were cavities and watercourses in the rock itself where two strata came together and in fact the rock near the surface was literally honeycombed with holes and channels. These were in some cases filled with a soft gray clay and in others were entirely clean and in the latter event they would run water. The water coming from these seams or channels was clear like spring water while the river water was at all times muddy and turbid.

When it was found that the pumps would handle the water without trouble the air was taken off the caisson, and the working chamber allowed to fill with water. Also the timber of the crib walls was carried up to about elevation 92 and levelled up. The average water level at ordinary stages is about elevation 82. The space on top of the deck and between the walls of the caisson was filled with sand up to a point 1 ft. below the top excepting the upstream end where the filling was only carried up about 10 ft. above the deck and the remainder of the filling was done with concrete. One foot of concrete was also placed on top of the filling all the way around the caisson, bringing it up flush with the tops of the timbers. The sand filling was to give the crib weight and the concrete in the upstream end was to help withstand the blows and shocks from floating ice, while the concrete finish on top of the sand filling was to prevent the water from washing out the sand when it was high enough to run over the top of the crib. Timber heads were bolted to the upright timbers and left projecting above the concrete finish and eye bolts were placed in the sides of the crib to provide mooring places for barges.

CONCRETING OPERATIONS.

All concreting both for the top and inside the working chamber, was mixed by hand, the gravel, sand and cement being loaded on a barge with a clear space left on the deck on which to mix. The concrete was shovelled into a bottom dump bucket and swung to the top of the crib by a derrick barge. When concreting in the working chamber the concrete was sent in through a concrete lock placed on top of the supply shaft nearest to where it was needed. There were four of these supply shafts in the caisson as shown in Fig. 1, and each of them was supplied with a bottom door of steel plate and with a long key to hold it shut.

About 10,000 sacks of clay were deposited outside the cutting edge previous to starting work inside the caisson. To keep these sacks from being washed down stream by the current while they were being placed they were dropped through about 24 ft. of 24-in. supply shaft which was swung on a derrick and held with the lower end slightly above the river bottom. Several sacks were thrown down this tube after which it was raised slightly and others thrown in. When enough had been deposited in one place the tube was shifted along. By means of a sounding rod it was possible to keep close track of the location of the sacks.

When the area inside of the cofferdam was cleaned out it was found that the rock bottom was from elev. 62 to elev. 64. This had to be excavated to elev. 61 and lower if the rock was not considered suitable at that elevation. It was finally excavated to about elevation 59 and cleaned up ready for the masonry of the intake tower.

The rock was drilled with Ingersoll-Rand Jackhamer drills and shot with very light charges of 40 per cent dynamite as it was desired to shake the crib as little as possible and at the same time it was necessary to leave a perfectly sound stratum of rock for the foundation of the tower. The shooting merely loosened the rock after which it was wedged and pried up, loaded in scale boxes, hoisted by the derrick barges and dumped on the decks of barges. These barges were then towed ashore and unloaded.

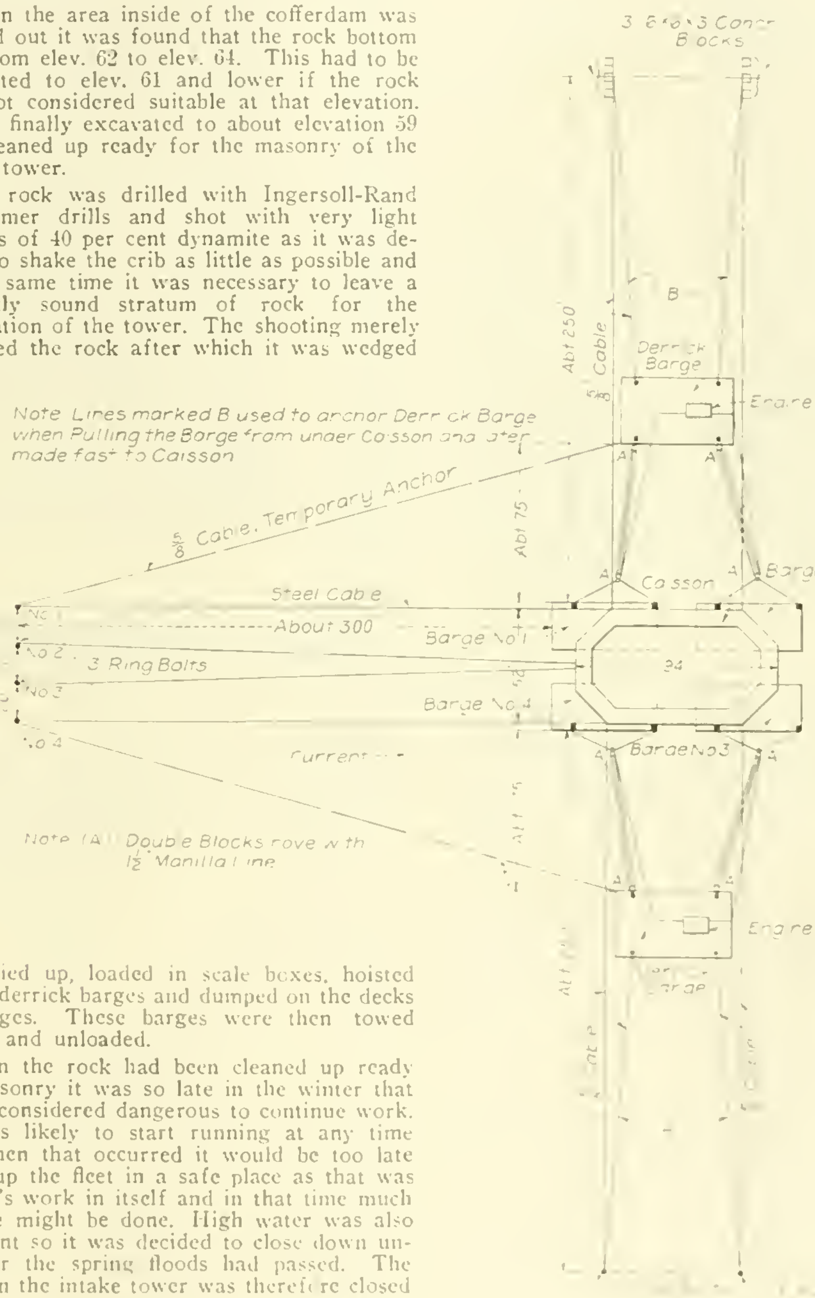

3 6×6×3 Conc Blocks
B
Abt 250'
5/8" Cable
Derrick Barge
Engine
A
Note Lines marked B used to anchor Derrick Barge when Pulling the Barge from under Caisson and after made fast to Caisson
5/8 Cable, Temporary Anchor
Abt 75'
Caisson
Barge No 2
Steel Cable
No 1
About 300
Barge No 1
No 2
3 Ring Bolts
34
No 3
Barge No 4
No 4
Current
Barge No 3
Note (A) Double Blocks rove with 1½" Manilla Line
Engine
Derrick Barge

Fig. 11. Sketch Plan Showing Location of Caisson Anchors and Arrangement of Barges and Lines for Launching Caisson.

When the rock had been cleaned up ready for masonry it was so late in the winter that it was considered dangerous to continue work. Ice was likely to start running at any time and when that occurred it would be too late to tie up the fleet in a safe place as that was a week's work in itself and in that time much damage might be done. High water was also imminent so it was decided to close down until after the spring floods had passed. The work on the intake tower was therefore closed down Dec. 29, 1913, and by Jan. 6, 1914, all plant had been removed from the crib and from the power barge and all the floating equipment had been laid up in winter quarters.

CONSTRUCTION PLANT.

The plant used on this work consisted of a power barge 32x130 ft., on which were three 60 HP. locomotive type boilers carrying 110 lbs. steam pressure, one 675 cu. ft. per minute capacity straight line Ingersoll compressor, a 7½KW. direct connected Allis-Chalmers lighting set (110 volt), a Smith 1 cu. yd. concrete mixer intended for use on the tower proper but which was not used on the caisson work, and a duplex pump for furnishing cooling water for the compressor and for other purposes. On this barge was also a dry house for the use of the "sand hogs" and other workers. The compressor was used for low pressure air for the caisson and later on was used for supplying high pressure air to run the rock drills. The boilers supplied steam for the compressor and for all the pumps. The power barge was moored at the downstream end of the caisson and made fast to it with steel cables. Steam and air lines on the crib and on the barge were connected through rubber hose in the case of the air and flexible metallic hose for the steam.

Two derrick barges were also used, one being the high derrick with which the anchors were set. On it was placed a Westinghouse compound air compressor with a capacity of 150 cu. ft. per minute, which supplied air for the grouting and also for the drills part of the time. The other derrick barge was a combined stiff leg and A-frame derrick with 32-ft. mast and 50-ft. boom set on a 30x60-ft. barge. In addition to the above five material barges were in use and also one steamboat and two gasoline launches for handling men and material back and forth.

This work is being done for the St. Louis water department. E. E. Wall is water commissioner, Gordon Black is chief engineer and E. C. Davis is resident engineer. The contractors for the work are The Fruin-Colnon Contracting Co. and the Missouri Valley Bridge and Iron Co. jointly, with C. H. Hollingsworth as general superintendent and W. Naugle, R. E. Mosier and W. B. Converse as assistants on the various operations.

Inspecting, Coating and Testing Steel Pipe for 112 Mile High-Pressure Gas Line from Taft to Los Angeles, Calif.

The construction of a 112 mile high-pressure steel pipe natural gas line from Taft to Los Angeles, Calif., was completed in the fall of 1912. The following notes on the mill inspection of the pipe and the field coating and testing of the pipe line are taken from an article by W. E. Barrett in a recent issue of the American Gas Light Journal.

The pipe line was designed to work at an initial pressure of 450 lbs., and a terminal pressure of 50 lbs., which would give a delivery capacity of 26,000 million cubic feet per day. The pipe ordered was plain-end steel tubing, 12¾ ins. outside diameter, weighing nominally 33 lbs. per lineal foot. About 75 miles of this pipe was of double length, ranging from 34 ft. to 40 ft. in length; the balance was standard, 18 ft. to 20 ft. lengths. By using the double lengths of pipe, in the valleys on the north and south ends of the line, a saving of approximately $31,000 in the cost of couplings alone was effected for the benefit of the client, as the work was carried on under the J. G. White Engineering Corporation standard method of "Cost plus a fee."

Inspection.—As this pipe line was to work under heavy pressure, it was deemed advisable to exercise very reasonable precaution in its manufacture and preparation for shipment as well as in the construction of the line. The J. G. White Engineering Corporation furnished its own retained inspectors at the mills of the contractor tube company. The certificates of chemical composition of the ingots from which the skelp was made were at the inspectors' disposal, upon request. The inspectors' principal duties were to observe the size by the ring test, the alignment of the tube, the length, weight and hydrostatic pressure test; this latter was required at 700 lbs. While the pipe was at a temperature of practically 100° F., brushed free from all mill scale, it was given a shipping coat of specially prepared protective material, allowed to dry, then shipped.

Coating.—On account of the large percentage of ground salts (strongly alkaline in composition) present along the proposed route, the protection of the pipe from the destructive effects of these salts was a very necessary precaution. For this purpose, a carefully prepared, neutralized coating was used, which had for its base a clean coal tar not a tar produced at a low temperature of say 1,600° F., to 1,800° F., but one produced at a temperature of about 2,400° to 2,600° F. There is a considerable difference even in coal tar, outside of the fact that it is black. After the pipe was received and distributed, any abrasions of the shipping coat were carefully cleaned and all dust or dirt brushed off. The spots were first coated, then the entire length given two, three or four coats, as the analysis of the soil dictated. These analyses were made in the laboratory of the University of Southern California.

Testing.—After the line was completed and the pressure tests were applied, only one length was found defective, and in this length the seam ripped at 368 lbs. The line was tested to 470 lbs., for the first 43 miles from the Taft terminal; 370 lbs., for the next 39 miles; and 250 lbs. for the balance of the distance to the Los Angeles terminal, and, after tightening some of the asbestos joints, with which the first 10 miles of the line from the Taft terminal was laid, to the small whisper-leaks, which only developed after the line had been raised in pressure to 368 lbs., then dropped to zero, when the blowout occured, and again raising the pressure to 470 lbs., careful inspection showed the small leaks. These leaks were apparently caused by the lack of sufficient resiliency in the gaskets to recover their original position after the high internal pressure had been applied. However, when these were once made tight under pressure they remained permanently so. The line when finally tested proved bottle tight. A second test was applied, about May 20, 1913, after the line had laid over the wet season. A pressure of 170 lbs. was then applied, from end-to-end, and it was still perfectly tight, holding this pressure 7½ hours without any drop in the gages. All the gages were carefully proven on two dead-weight test sets, and adjusted to perfect agreement.

Bridges

Design and Construction Features of the Tunkhannock Creek Viaduct on the D., L. & W. R. R. at Nicholson, Pa.

Contributed by C. W. Simpson, Resident Engineer D. L. & W. R. R., in Charge of Construction.

The Delaware, Lackawanna & Western Railroad Co. is constructing some grade improvement work of large magnitude between Clark's Summit and Hallstead, Pa., the improvement consisting of a new three-track line about forty miles long. The new line does not deviate widely from the present one, as can be seen by an inspection of the map shown in Fig. 1. The greatest divergence is about 1½ miles, but for the greater part of the distance the lines are only a few hundred feet apart. The new line does, however, greatly reduce the grades and eliminates the sharp sags and peaks in the grade line. The maximum grade has been reduced from 1.23 to 0.682 per cent, and a very long 0.40 per cent grade has been reduced to a 0.237 per cent grade. The principal structures along the new line are the Nicholson Tunnel, a double-track bore 3,630 ft. long; the Martins Creek Viaduct, a three-track concrete structure having a maximum height of 150 ft. and a length of 1,600 ft.; and the Tunkhannock Creek Viaduct, a double-track structure having a maximum height of 240 ft. and a length of 2,375 ft. The quantities of materials involved in the construction of the new line are 7,617,000 cu. yds. of rock, 5,525,000 cu. yds. of earth, 116,000 cu. yds. of tunnel excavation, 370,000 cu. yds. of concrete, and 2,360 tons of reinforcing steel. The total cost of the improvement will be about $12,000,000.

This article will treat only of that part of the improvement known as the Tunkhannock Creek Viaduct, the construction features of this viaduct being explained in detail.

DESIGN FEATURES.

The Tunkhannock Creek Viaduct, which is said to be the largest concrete railroad bridge in the world, is composed of ten 180-ft. and two 100-ft. semi-circular arch spans springing from solid piers founded on bedrock, supporting transverse spandrel walls, upon which rests a floor system composed of semi-circular spandrel arches. The deeper piers have a section 40x46 ft. below the ground surface, and all piers are 36 ft. 6 ins. by 43 ft. 6 ins. to a point 17 ft. 6 ins. below the springing line of the arches. At this elevation a 4-ft. 3-in. offset provides a seat for the temporary steel centering. The deepest pier extends 98 ft. below the ground surface, and at this pier it is 304 ft. from the bottom of the foundation to the highest point of the masonry.

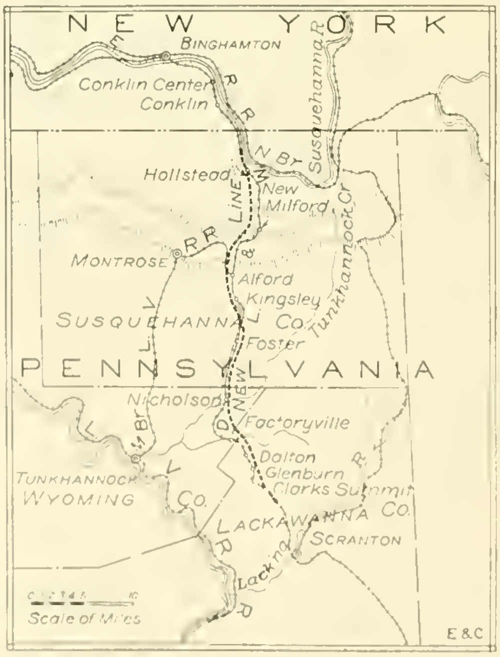


Fig. 1. Map of Region in Vicinity of New Line of D., L. & W. R. R., of Which Tunkhannock Creek Viaduct Is a Part.

A 100-ft. span is located at each end of the structure. This is termed an "abutment span," as it will be entirely buried by the approach embankment. Each of these spans is composed of two ribs 5 ft. 6 ins. thick at the crown and 12 ft. wide. These ribs are 22 ft. on centers, thus leaving a 10-ft. space between the ribs. The two ribs are tied together by four reinforced concrete struts. They support reinforced transverse walls on which rests a reinforced floor slab varying from 1 ft. 9 ins. to 2 ft. 6 ins. in thickness. The floor slab extends a few feet beyond the crown of the arch, and the space between the arch ring and the floor slab is closed with an 18-in. longitudinal curtain wall along each outside face, so that when the approach embankments are completed these "abutment spans" will have the appearance of "U" abutments.

Each 180-ft. span, a design drawing of which is shown in Fig. 2, is composed of two ribs 8 ft. thick at the crown and 14 ft. wide. These ribs are spaced 20 ft. on centers, thus leaving a 6-ft. opening between them. Each rib is reinforced with fourteen 1-in. square steel rods in both the intrados and the extrados (see Fig. 2, b and c). These ribs carry reinforced transverse spandrel walls varying from 3 ft. 2 ins. to 4 ft. 6 ins. in thickness (see Fig. 2, a and b). The spandrel walls are arched over the space between the ribs and carry a floor system composed of 13-ft. 6-in. semi-circular spandrel arches having a thickness of 1 ft. 9 ins. at the crown. The spandrel walls carry a 6-in. belt course 1 ft. 6 ins. below the springing line of spandrel arches, thus providing a seat for the temporary arch centers.

Directly over the piers the span opening which is left between the segments of the arch is closed by means of a pilaster 3 ft. thick and about 22 ft. wide, bonded to the spandrel walls to give longitudinal stiffness to the spandrel system and an appearance of solidity to the piers. The parapet wall, which extends the entire length of the viaduct, rises 7 ft. 3 ins. above the viaduct floor (see Fig. 2, b and c). The pilasters at the piers extend 3 ft. above the top of the parapet wall (see Fig. 2, b and d). To provide drainage the top of the floor is given a 6-in. slope from the crown of each spandrel arch to the center of the adjacent spandrel wall, and the water is carried down through the center of the wall through a 6-in. opening and is discharged into the space between the main arch ribs. The floor over each span is provided with four expansion joints, one over each main pier and one over

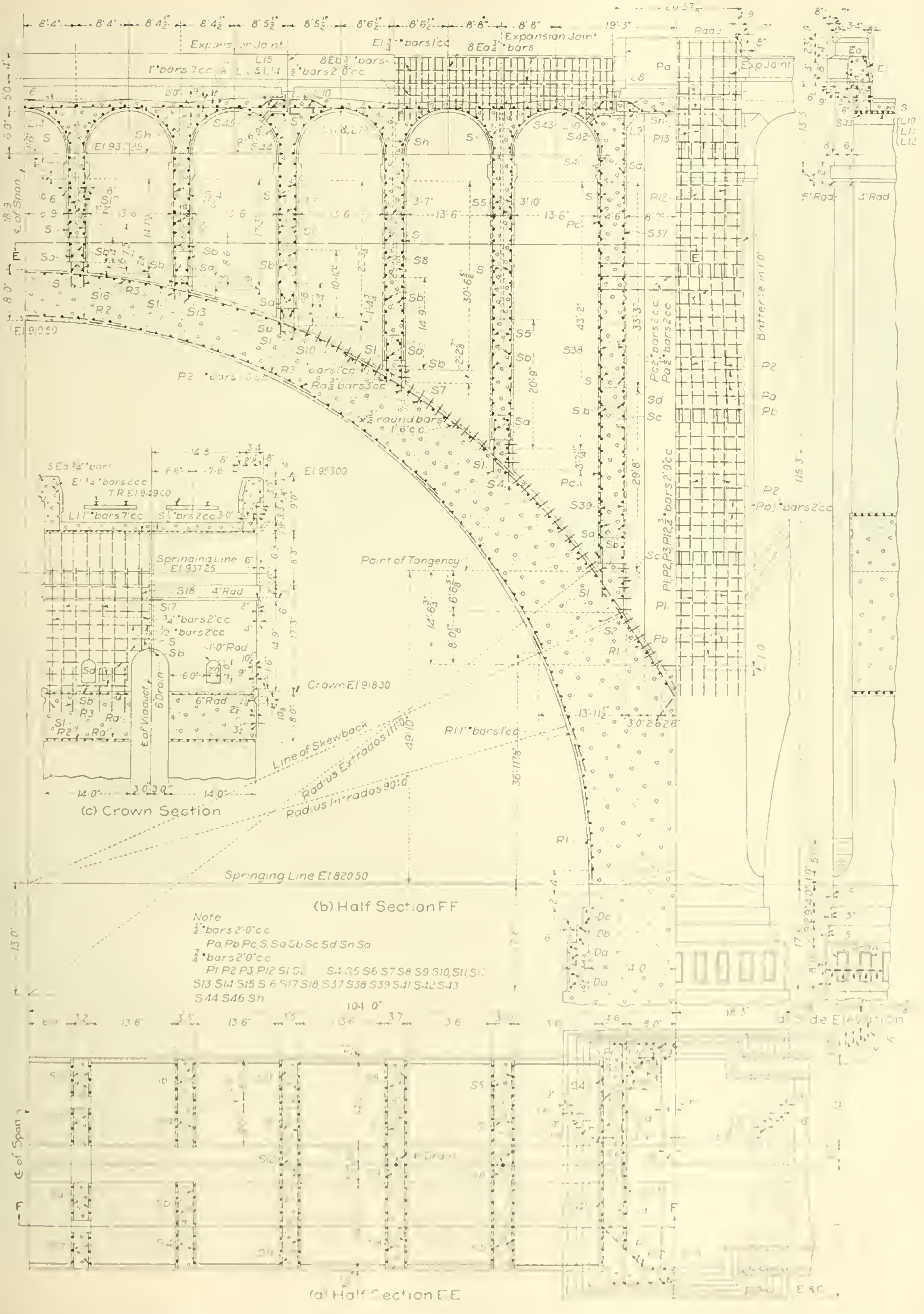


Fig. 2. Plans, Elevation and Sections of 180-Ft. Reinforced Concrete Arches of Tunkhannock Creek Viaduct.

the third spandrel wall from the pier (see Fig. 2, b).

This viaduct requires 47,000 cu. yds. of foundation excavation, 165,000 cu. yds. of concrete and 1,200 tons of reinforcing steel. Three classes of concrete were used, designated as "Class A," "Class B," and "Cyclopean." Class A concrete is a 1:2:4 mixture and is used only for the reinforced floor slab over the "abutment spans"; Class B is a 1:3:5 mixture, and is used for all other work above the springing line; "Cyclopean" is the same as Class B, with "derrick" stone embedded in the concrete, and is used in the piers below the springing line.

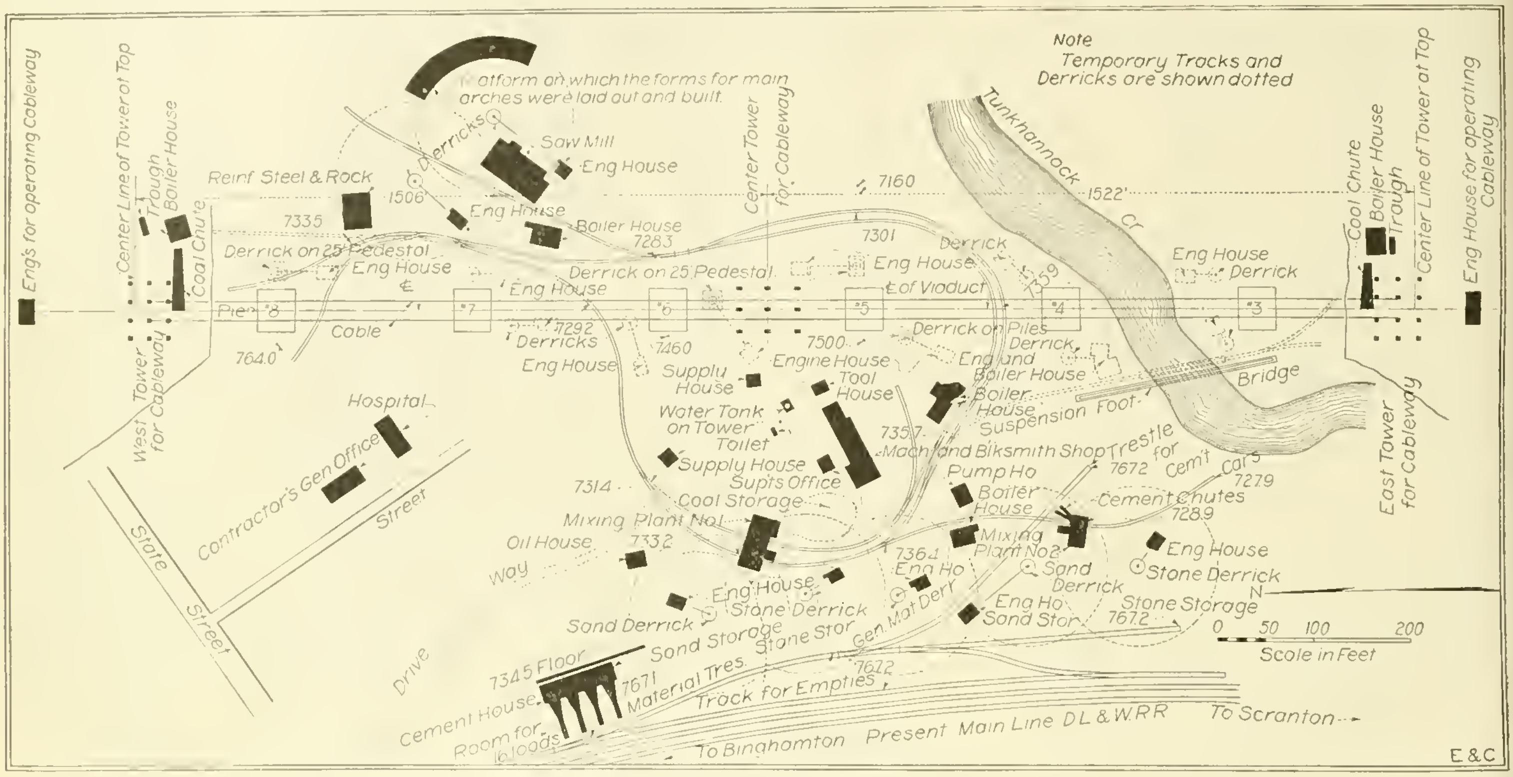


Fig. 3. Layout of Contractors' Plant for Construction of Tunkhannock Creek Viaduct.

CONTRACTOR'S PLANT AND GENERAL PROCEDURE.

The contract for the construction of the viaduct was formally executed in June, 1912, but the successful bidder was notified about the middle of May and started work on May 21, 1912.

The general layout of contractor's plant is shown in Fig. 3. The present tracks of the railroad company are on an embankment which rests on a small ridge, so that they are about 60 ft. higher than the creek bed and about 35 ft. higher than the ground level at the mixing plants. The loop formed by the narrow-gauge tracks encloses a knoll about 30 ft. higher than the general level of the valley. The general office and the hospital are located on higher ground.

A narrow ridge extending westward from the west end of the blacksmith shop connects the above mentioned knoll with the ridge on which the railroad company's tracks are located. The general material derrick is located on this ridge. A similar ridge extends from pier No. 6 to the high ground at the hospital. Piers Nos. 5 and 6 are located on the east slope of the knoll so that the ground surface at the west side of the excavations is about 30 ft. higher than on the east side. Piers Nos. 3, 4 and 7 are located on the low ground and pier No. 8 lies on a slope, so that the ground surface at the south side of the excavation is 10 ft. lower than on the north side.

The first operation of the contractors was the widening of the railroad company's embankment and the laying of the track marked "Track for Empties" and "Room for 16 Loads" (see Fig. 3). This work was started May 21, 1912, and on June 12 the track was completed and a track piledriver started to construct the material trestles. The general material derrick was erected in June, during which time work was progressing on the shops, storehouses and office buildings.

FOUNDATION EXCAVATION.

After the narrow-gauge tracks had been graded and laid from the location of mixing plant No. 1 to pier No. 4, embracing a small cut through the above mentioned ridge, derricks were erected, steel sheet piling was brought in, and excavation was started early in July at pier No. 4.

On July 16 a "Model 40" Marion steam shovel, having a 1-cu. yd. dipper, was brought in and started digging its way through the ridge connecting pier No. 6 with the high ground at the hospital. The material excavated was loaded on 4-cu. yd., 3-ft. gauge Western dump cars, and was utilized in grading for

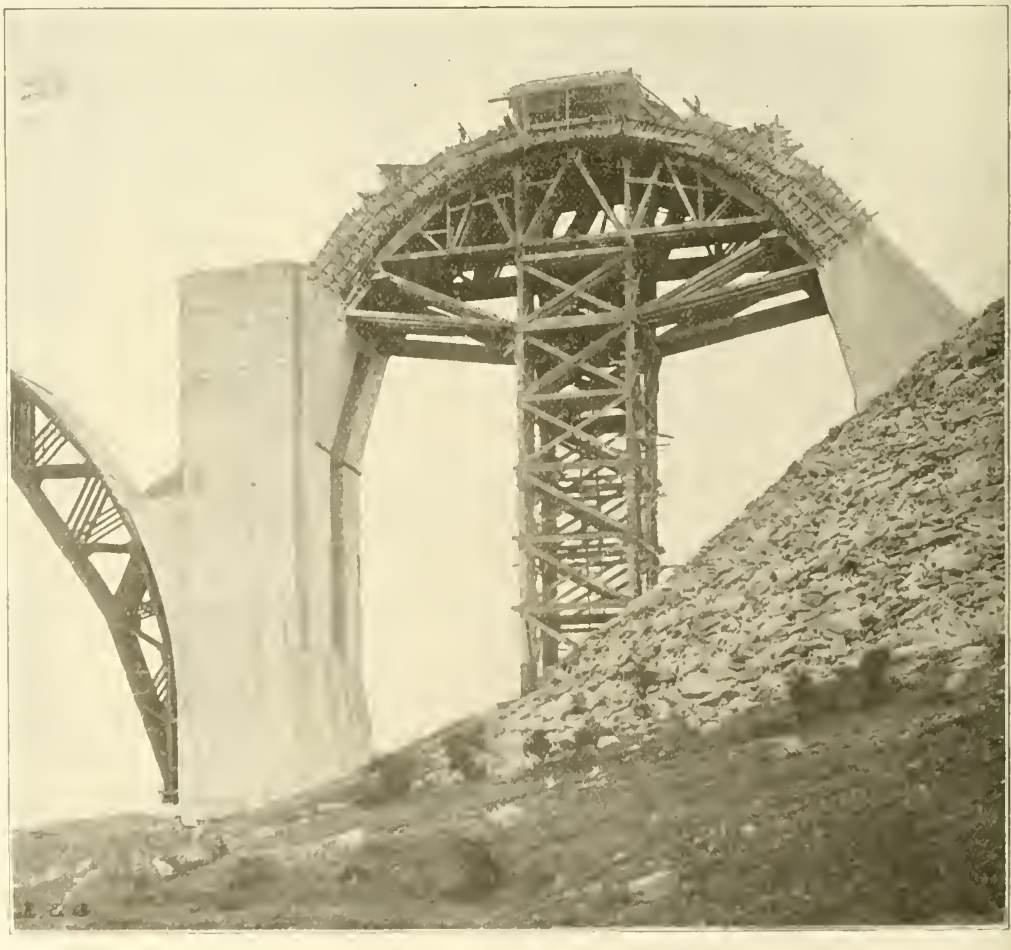


Fig. 4. Wooden Centering and Support for 100-Ft. End Arches of Tunkhannock Creek Viaduct.

the narrow-gauge tracks and for leveling the ground surface around the mixing plants. This shovel excavated pier No. 6 to a level 20 ft. below the ground surface on the high side, then moved to pier No. 5 and excavated this pier down to the level of ground water, which was about 35 ft. below the ground level on the high side. From pier No. 5 the shovel moved back to pier No. 6, and after excavating this pier to the same level as No. 5 moved on to pier No. 8, where the excavation was carried to the same level. The material from these excavations was utilized in raising the entire ground surface east of piers Nos. 7 and 8 a distance of about 12 ft., thus bringing it up to a level with the surface graded around the mixing plants and providing a satisfactory location for the saw mill and for the storage of reinforcing bars, lumber and other materials. From pier No. 8 the shovel was taken successively to piers Nos. 10, 9 and 2, in each case excavating the material to bedrock, which was above the level of ground water and was only a few feet below the surface at the lower sides of the piers. In making these excavations the material, which was a mixture of clay, gravel and sand, was allowed to take its natural slope, and the excavation was extended far enough beyond the sheeting lines to provide for the natural raveling of the banks prior to the completion of the excavation. The shovel completed its work in December, 1912, after having excavated about 50,000 cu. yds., 15,000 cu. yds. of which was "pay material."

In excavating piers Nos. 3, 4 and 7 and in continuing the excavation of piers Nos. 5, 6 and 8 the contractor used Lackawanna interlocking steel sheet piling in lengths of 30 ft. This piling was braced with 12x12-in. timbers, and was driven with a three-ton Warrington steam hammer. As these piers required sheeting for depths of from 50 to 70 ft, it was necessary to use two lengths of sheeting, and in two cases short lengths were driven on top of the upper 30-ft. lengths. The method of procedure with the excavation was to erect one set of sheeting on lines about 3 ft. outside of the required foundation area, and to drive the sheeting as far as convenient. The excavation was then carried down to the bottom of the sheeting by means of Williams 1½-cu. yd. clam-shell buckets, operated in the bays between timber braces which were so spaced as to admit the operation of the bucket. The bracing was then put in on about 5-ft. centers, vertically, as the excavation proceeded. After the excavation has been carried down to the temporary bottom of the sheeting, the latter was driven farther, and the operation was repeated until the top of the sheeting was down to the ground surface. Another set of sheeting was then erected about 6 ft. 6 ins. outside the first set, and was driven as deep as was thought advisable. The material between the two sets of sheeting was excavated with the clam-shell bucket, the first sheeting was then driven farther, and the excavation was resumed. As the inside set of sheeting was driven down, the upper set of timber bracing was removed and placed at the bottom. The process was continued until bedrock was reached. In two cases the two 30-ft. lengths of sheeting were not sufficient and short lengths of sheeting was driven directly on top of the second set. The contractor has been able to recover almost all of the sheeting, so that sufficient sheeting for two complete foundations has sufficed for the entire work. Some of the sheeting has been used four times. All of the 30 ft. sheeting was purchased new for this work, but the shorter lengths had been previously used on other work. The sheeting was pulled by means of an "A" frame, constructed of 12x12-in. timbers about 16 ft. long, resting on the top of the completed pier footing and carrying a triple block-and-fall rove with a ¾-in. steel cable. This fall was attached to the sheet piling by means of a bolt through the web, and a rasp clamp. The derrick fall, which is a double block, was made fast to the ¾-in. cable, and in this way a pull of about 60 tons could be exerted on the pile.

The material excavated from pier No. 4 was utilized to grade the track shown by dotted lines in Fig. 3, and a trestle was constructed across the creek, thus giving trackage facilities to pier No. 3, the excavation of which was started early in September. The excavations for the abutments and the piers near the ends of the viaduct were shallow, and the work was accomplished without the use of derricks or sheeting. The only excavation which gave any serious trouble was for pier No. 4. About 40 ft. below the surface there was encountered a large pocket of fine flowing sand along the west side and around the southwest corner of the pier. This sand exerted a very heavy pressure on the sheeting, distorting the timber bracing and several times breaking up through the bottom of the excavation and causing the ground surface outside the sheeting to settle as much as 10 ft. When the excavation had reached a depth of 50 ft below the surface and was about 12 ft. above bedrock it was found that very little progress was being made, the bracing being so badly distorted as to be in grave danger of failure. At this point it was decided to drive a line of 3-in. plank sheeting parallel with the west side of the pier and about one-third of the width of the excavation from it. The portion of the excavation to the east of this sheeting was then carried down to rock, and was filled with concrete to a depth of 30 ft. All timber bracing was left in place, thus making any further distortion of the west one-third almost impossible. A row of 3-in. timber sheeting was then driven on an east-and-west line across the remaining one-third of the excavation. The northwest corner was then excavated, and concrete was placed to a depth of 10 ft. The southwest corner was now excavated and concreted without serious trouble. The excavation of this pier was not completed until the middle of July, 1913. All of the foundation excavation is now completed, with the exception of that required for pier No. 5, which is about 30 ft. above bedrock.

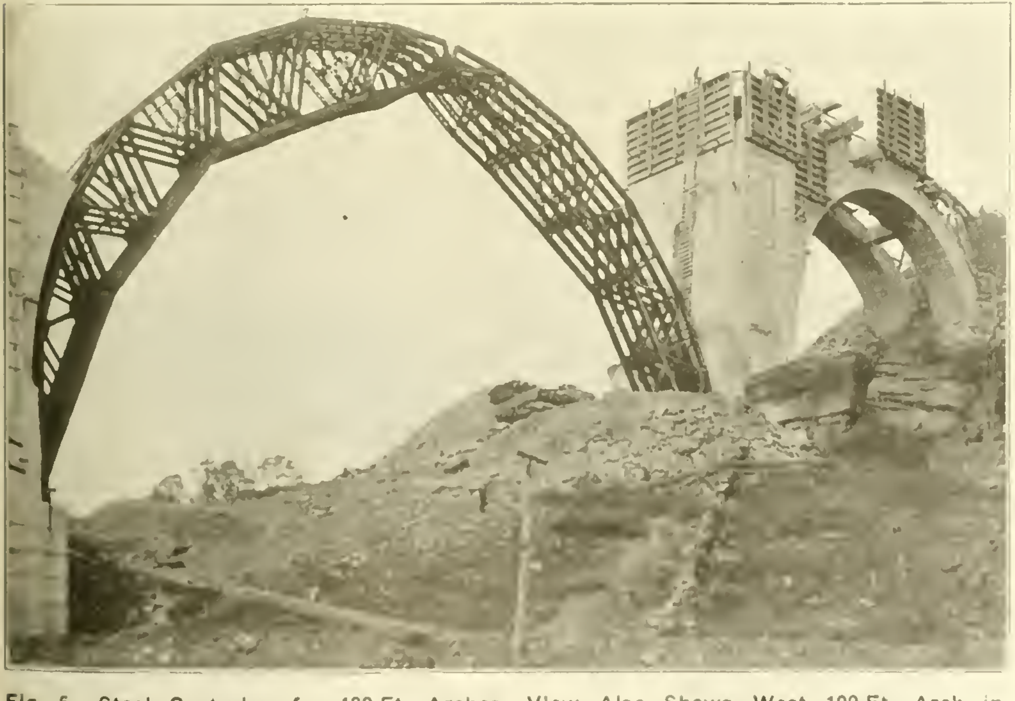


Fig. 5. Steel Centering for 180-Ft. Arches—View Also Shows West 100-Ft. Arch in Process of Construction.

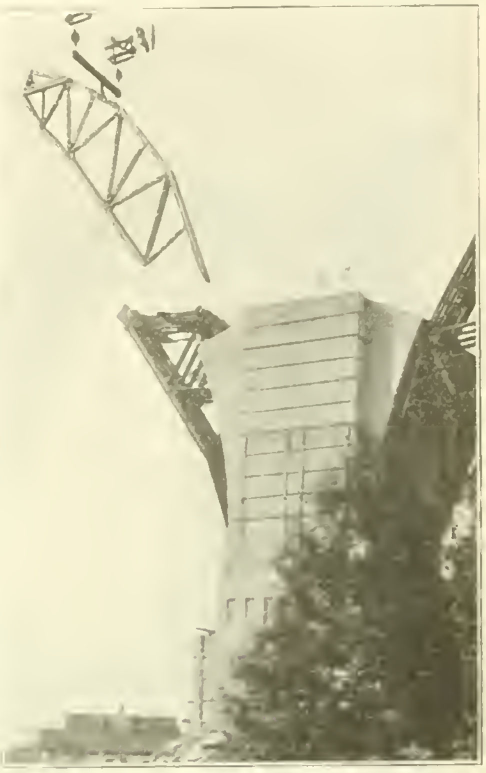


Fig. 6. Erection of 180-Ft. Steel Centers for Intermediate Spans by Means of Cableway, Tunkhannock Creek Viaduct.

CONCRETE PLANT.

Prior to January, 1913, the cement shed, which has a capacity of 3,000 bbls., had been completed and filled, mixing plant No. 1 had been completed (for location see Fig. 3), and a large amount of sand and stone had been placed in the storage piles. The sand and stone are placed on the contractors' side track in bottom-dump hopper cars by the railroad company. The cars are spotted on the material trestle, which is about 30 ft. high, by a 30-ton standard-gage locomotive and their contents dumped into the storage piles. Each mixing plant is served by two derricks which operate 40-cu. ft. Meade-Morrison clam-shell buckets. At mixing plant No. 1 the cement is ordinarily loaded directly from the cars into the derrick bucket, and is landed on the hopper floor of the mixer. At plant No. 2 the cement is unloaded from the cars into chutes which convey it by gravity to the hopper floor of the mixer. Cement is taken from the cement house only when shipments are delayed and when there is no cement available in cars. The mixer derricks convey the sand and stone from the storage piles to small bins over the mixers. From these bins the materials pass by gravity into a measuring hopper where the cement is added. From the measuring hopper the materials pass by gravity into a Carlin cube mixer of 2 cu. yds. capacity, which dumps directly into 2 cu. yd. Lockwood double-line bottom-dump buckets on 3 ft. gage flat cars. Twelve-ton locomotives are used to haul trains of three cars (one empty to receive the empty bucket from the derrick or cableway) to the point of disposition or to the point of transfer to the cableway.

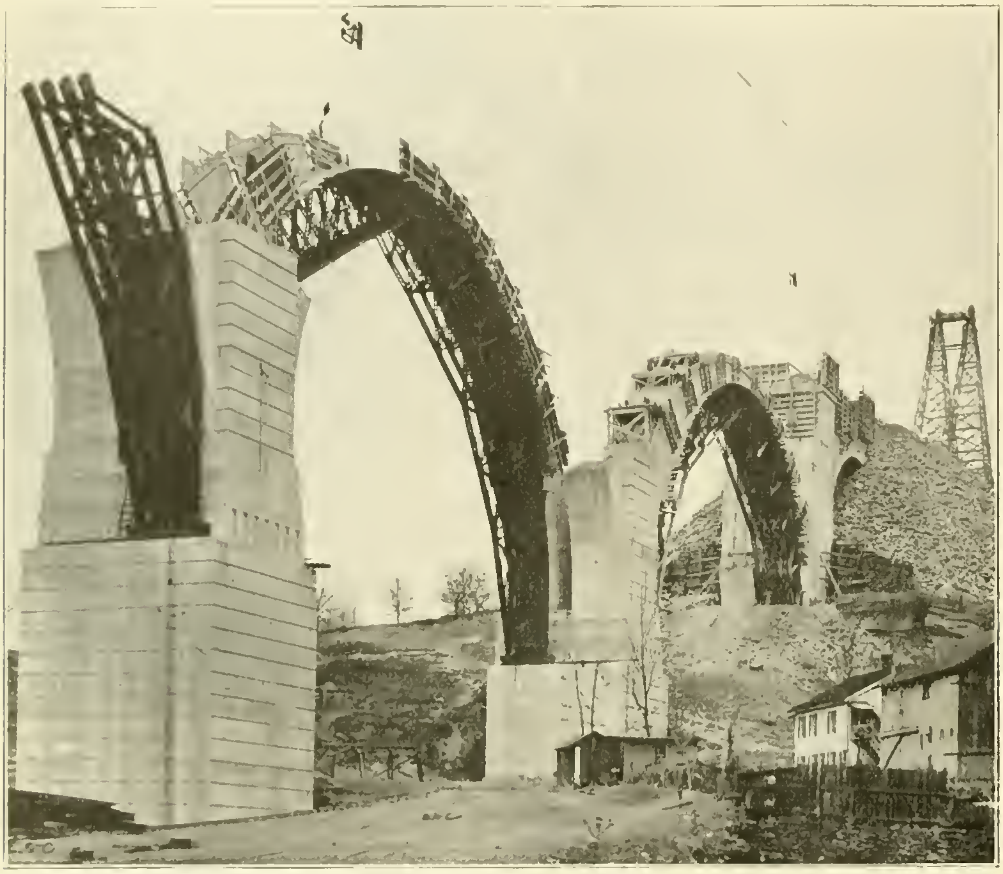


Fig. 7. West Spans of Tunkhannock Creek Viaduct, Showing Centering and Forms—One Rib Ready for Running Keys—Abutment Span Partially Buried.

The contractors are required to turn the mixer for not less than two minutes, and, although adhering to this requirement, each mixing plant has a regular working capacity of 18 batches of 2 cu. yds. per hour, about one minute being consumed in charging, starting, stopping and dumping the mixer. One foreman, 2 derrick operators, 1 mixer runner, 1 fireman, and 11 laborers are required to operate one mixing plant; while 1 foreman and 8 laborers, in addition to the engineman and helper, are employed to unload all cement, sand, and stone for both mixing plants. These gangs, with the aid of the material derricks, also unload practically all other materials and supplies used on the work. Two trains are used to transport the concrete from the mixer to the derrick or cableway. It requires 2 men to hook the buckets, 2 operators for the derrick or cableway, and 2 signal men to place the concrete in the forms where it is spread by 1 foreman with from 8 to 16 men. In the massive concrete work below the springing line of the main arches, this organization handles about 35 cu. yds. of concrete per hour.

A bag room, built as a part of the mixing plant, is provided. The floor of this room is about 8 ft. lower than the hopper floor, and as soon as the cement bags are emptied into the hopper they are thrown into this room. One man carefully shakes the cement from these bags, and ties them into bundles ready for shipment back to the mills, so that when the mixer stops at the end of a day's work the bags used that day are all ready to return. The contractors state that the loss of bags is less than 2 per cent, and that of this loss a considerable portion has been used for legitimate purposes on the work. Although more than ordinary care is taken to empty thoroughly the bags into the material hopper, test measurements of the amount of cement regained on the bag-room floor show that this saving more than pays for all the expense connected with the handling and shipping of the empty bags.

The first concrete was placed on Jan. 6, 1913, and as the winter of 1913-14 was very mild concreting has been carried on almost continuously up to the middle of February, 1914. The total amount of concrete placed during this time was about 90,000 cu. yds., the largest amount being placed during the month of June, 1914, when 11,000 cu. yds. were put in place, about 9,000 cu. yds. being placed in forms. The percentage of the work completed at the end of each month from January, 1913, to February, 1914, was as follows:

Date.	Percentage completed.
January, 1913	5.0
February	8.4
March	11.1
April	15.0
May	19.2
June	27.5
July	34.7
August	39.3
September	45.1
October	47.4
November	49.3
December	50.6
January, 1914	52.6
February	53.9

FORMS FOR PIERS AND ARCHES.

The design of this bridge is such that sectional forms can be used with great economy and the contractors are taking advantage of this feature to an unusual extent. With few exceptions the forms are built on the ground, in sections, and are raised to the required position by the derrick or cableway. As an illustration of the utility and construction of these sections, the forms used for the main pier shafts will be described somewhat in detail. These forms are of two sizes, one of which is 15 ft. 8 ins. long by 17 ft. 9 ins. high, and the other, 18 ft. 3 ins. long by 17 ft. 9 ins. high. Two of the longer sections form the shorter sides of the pier, and three of the shorter ones form the long sides. Ten sections are required to surround one pier, and they provide for concreting a net height of 16 ft. These ten sections constitute one set, and four sets have sufficed for concreting all of the piers up to the centering ledge, without in any way retarding the progress of the work. One set is being retained for completing piers Nos. 5 and 6, and the other sections are now being used with little or no alteration for other parts of the work. Each section is made by nailing two layers of 1x8-in. tongue-and-groove boards to 8x10-in. horizontal studs spaced 2 ft. 5 ins. on centers, one layer being placed at right angles to the other, both layers being placed at 45° with the studs. These studs are bolted to 10x10-in. verticals, spaced so that a vertical will be halved at each joint and one in the center of each section. The verticals extend about 2 ft. above the top of the planking. The form is faced with No. 26 galvanized sheet-iron, the larger section weighing about 7,000 lbs. The forms are kept from moving outward by rods which run down at an angle of 45° from the vertical posts to anchor bolts placed in the concrete; they are prevented from moving inward by 12x12-in. horizontal struts which rest on the top edge of the form and are wedged against the opposite verticals. The rods are left in the concrete, and are provided with a sleeve attachment so that the portion extending beyond the concrete face can be removed. Six carpenters with the aid of the derrick or cableway can remove and re-erect one set of forms in two days. A minimum cycle of operation frequently attained is as follows: Two days for concreting, three days for concrete to set, and two days for removing and re-erecting forms.

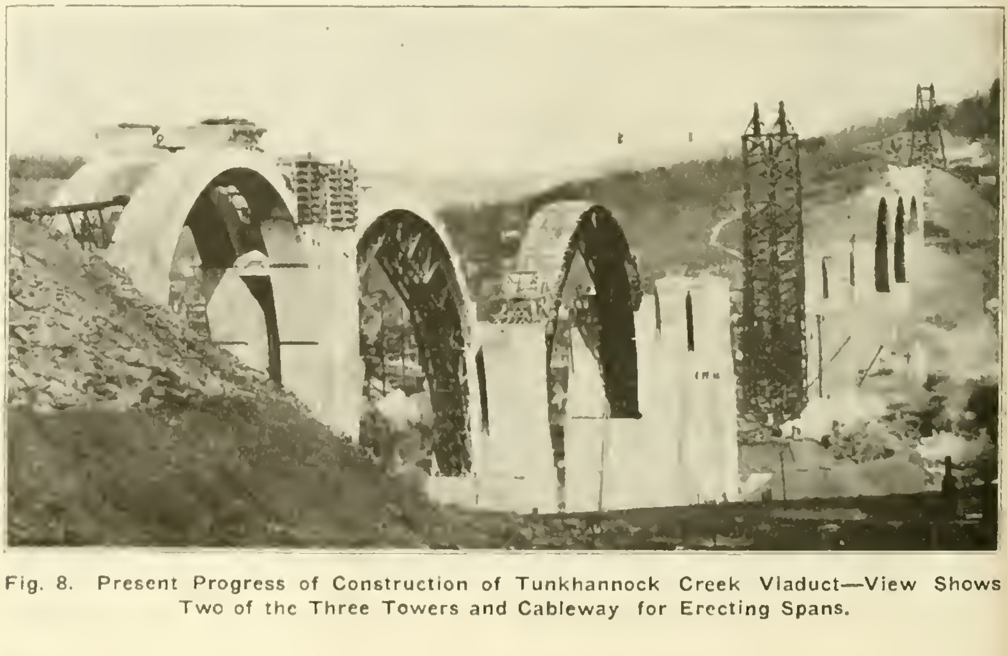


Fig. 8. Present Progress of Construction of Tunkhannock Creek Viaduct—View Shows Two of the Three Towers and Cableway for Erecting Spans.

In constructing the arch rings the concrete is being placed so as to form large voussoirs separated by small keyed openings. The voussoirs are all run and allowed to set for seven days before the keys are concreted. Fig. 7 shows some of the centers in place and a portion of the arch ring concreted. The forms for any voussoir are entirely independent of those for the adjacent blocks, so that any block can be concreted independently of any other. Separate forms are provided for closing the key openings, so that the voussoir forms can be used for concreting on one arch while the keys are being filled on another span. Two complete sets of arch forms have been constructed; they will suffice for the entire 20 main arch ribs.

The contractors' saw mill is equipped with circular and band saws, a planer, and an air compressor for the air-boring tools. The blacksmith and machine shop is equipped with forges, a planer, a lathe, a thread machine, and drill press.

ARCH CENTERS.

Wooden centers are used for the construction of the two end 100-ft. arches, the centers being supported on a wooden tower. Fig. 4 shows a view of one of the end 100-ft. spans with the centers and arch forms in place.

Five sets of three-hinged steel arch centers will be used in constructing the main arches, each set being composed of four ribs weighing 47 tons per rib, spaced 3 ft. 10 ins. center to center. Figure 5 shows a view of the steel centers for the 180-ft. arches. The view also shows the west 100-ft. span. Figure 6 shows a view of an intermediate pier with part of the steel centering for the adjacent spans in place. The view also shows the manner in which the steel centers are erected by the use of carriers supported on a cableway. Figure 7 shows some of the centers and forms at the west and of the viaduct, and indicates the order of constructing various parts of the work. These centers are supported on 4-ft. 3-in. ledges located 17 ft. 6 ins. below the springing line of the main arches. An I-beam grillage, resting on this ledge and extending the full width of the pier, supports roller nests of 6-in. rollers on which the pedestals of the centers rest. For erection purposes each rib of the centers was built in four sections. The centers are so designed that the half rib will support itself as a cantilever.

Before the centers are erected the piers are constructed up to a point about 37 ft. above the springing line, forming what is termed the "umbrella." The lower quarters are lifted into position, the bottom pin is driven, and the top of the section is anchored to the top of the umbrella by bolts which pass through the concrete. One of the remaining quarters is then raised to position and bolted to the section already in place. The last quarter is now raised into position, bolted to its adjacent lower quarter, and the crown pin driven. For striking the centers, the first panel on each side of the crown pin is constructed with pin connections, and the web member at right angles to the chords consists of two parts connected by a right and left thread screw operated by a lever and ratchet. By lengthening this member the distance between crown and end pins is shortened and the centers are thus lowered.

After one of the main arch ribs has been completed the centers are slacked down and are rolled over to their position under the twin rib. When both ribs are competed the centers are rolled back under the opening between the ribs and are taken up through this opening and transported to their next point of service.

The centers were built by the Fort Pitt Bridge Works.

CABLEWAY AND DERRICKS.

All derrick timbers were shipped to the work in a rough condition, and were dressed, framed and fitted by the contractors. Castings were made in commercial foundries from patterns owned by the contractors, and other fittings were made in the machine shop. The masts are from 85 to 90 ft. long, and the booms are from 80 to 85 ft. long.

All derricks are operated by three-drum, 8¼x12-in. Mundy hoisting engines with swinging gear attachment. In a few cases the derricks have been placed on timber pedestals about 25 ft. high, but in most cases they have been erected at the ground level, and all work above the reach of the derricks is carried on by means of a cableway.

The erection of the cableway was started in August, 1912, and it was put in operation in February, 1913. There are two lines of 2¼-in. main cables spaced 20 ft. apart on centers and supported by end towers 150 and 165 ft. high and by a center tower 260 ft. high. The head blocks of the end towers are 3,028 ft. apart, and the included distance is divided by the intermediate tower into two spans of about equal length. Four engines and carriages operate as four independent units. The towers are of timber, and the type of construction is indicated in Fig. 8. The view also shows a part of the cableway and indicates the present state of construction of the viaduct.

The cableway was designed by and all material, except timber, was furnished by the Lidgerwood Manufacturing Co. The contractors for the viaduct built the towers and erected the cableway.

PERSONNEL.

The bridge was designed and is being constructed under the direction of the Engineering Department of the Delaware, Lackawanna & Western Railroad Co., of which Mr. G. J. Ray is chief engineer, Mr. F. L. Wheaton is engineer of construction, Mr. A. B. Cohen is concrete engineer, and the writer is resident engineer in charge of the viaduct. The construction work is being carried out by Flickwir & Bush, Inc., for whom Mr. F. M. Talbot is general manager, and Mr. W. C. Ritner is general superintendent.

Roads and Streets

Methods of Making Surveys and Plans for Trunk Line Highways in Michigan.

Under the Trunk Line Highway Law, which went into effect July 1, 1913, it became the duty of the Michigan State Highway Department to make the surveys and plans for the roads designated in the law. A paper describing some of the more important and interesting features of the field and office work involved and the way in which various difficulties encountered in the work were overcome was presented at the recent annual meeting of the Michigan Engineering Society by Mr. Harry L. Brightman of the Michigan State Highway Department. The information given in the paper is practical and of value to engineers who have road surveys and plans to make. Some of the methods are also useful for railroad surveying, especially for interurban railway surveys. We reprint the major portion of the paper as follows:

Office and Mapping Methods.—A set of drawing room standards was written and blueprinted in book form, which covers pretty well the details of the drawing. The plan shows a plat of the road with topography and drainage worked out. Projected below this is the accompanying profile showing the center line and ditch lines of the present ground together with two lines of hubs, one on each side of the roadway. There are three grades laid—the center line and each ditch. Five sets of figures are given showing the distance the center line grade is above or below the hubs and the actual cut or fill at center line from the present surface to the top of the finished metal surface. All this is put on a sheet of tracing cloth 36 ins. long by 20 ins. in width with 10x10 cross sections printed on it. This allows a scale of 100 ft. to the inch, natural, for the plans and 100 ft. horizontal and 10 ft. vertical to the inch for the profile, and gives a sheet to each half mile of road. There are usually two cross sections of the road shown on each sheet with suitable title and what detail information appears necessary.

The plan shows the curves to scale and the center line of the road is put always 2 ins. below and parallel to the top of the sheet. This necessitates breaking the plan, which is done without making a disconnected profile. For laying grades we have a long table which allows five sheets to be laid end to end and gives a continuous profile 2½ miles long on which we can lay a grade much more advantageously than on a single sheet at a time.

From the notes we lay as good a grade as we can in the office, figure the center line cuts and fills and blueprint the work. The prints are taken into the field, the grades are compared with the ground and the drainage checked up. The prints are then returned to the office together with the report of the inspecting engineer. The plans are altered to meet the suggestions in the report, and finished. They are blueprinted and sent to the township or county officials.

We limit the grade to 6 per cent for those roads which lie north of the line drawn from Bay City to Grand Rapids and to 4 per cent on roads south of that line except in a few special cases where it would entail a hardship on the county or township to cut to a 4 per cent grade. The rolling grade is thought by us to be the best. We use vertical curves for all changes of grade of 2 per cent or over. This enables us to fit the ground much better and to produce a grade which, when constructed, is both pleasing to the eye and easy for traffic.

For the purpose of making curves easier riding we give them a super-elevation in the following manner: Curves with a radius of 66 ft. to 250 ft. at a rate of 1 in. per foot; curves with a radius of 250 ft. to 600 ft at a rate of ¾ in. per foot; curves with a radius of 600 ft. to 800 ft. at a rate of ⅝ in. per foot; curves with a radius greater than 800 ft receive no super-elevation. The radius of curvature for a 7° curve is 819.02 ft. This is somewhat like the practice followed by the highway engineers of the state of New York. We differ from them, however, in that we believe the road should receive a crown when superelevated as well as when there is no superelevation.

Field Work.—The field work consists of the reconnaisance, the location survey and the inspection of the road during construction and upon completion. The location survey comprises running the center line, taking the topography, drainage, soil and material notes and running the necessary levels. This work is usually done by a party made up of a man sent out from this office who has charge of the party and keeps the notes, a local engineer and local men for chainmen, rodmen, etc. By this means we keep the surveys uniform in character and obtain the notes we want.

We encountered more difficulty with the field work than with the office work. The notes lacked many of the essential things and the work in the field was not conducted as it should have been at the start This was due, no doubt, to the lack of written instructions. It is difficult for a man to listen for an hour to a set of oral instructions and to go out in the field and carry them out in detail. To remove this difficulty a set of "Instructions to Field Men" were prepared. These instructions cover in a general way the work we want done in the field, and leave to the man's judgment the detail procedure for special cases. Some errors, and the most of these were made by local men, seem to be inexcusable, unless one can be excused for carelessness.

Many little, yet important, things were left out, such as names of small villages, streams, etc., the date of the work, the names of townships and section numbers, the description of the point of beginning, etc. Under the heading of "General" in the Instructions to Field Men are to be found instructions concerning the general make up of the field book. Section 8 states that "When crossing a township line the work in the new township shall start with Station 0 + 0.0." This makes it convenient to keep track of mileage of each township. Section 11 instructs field men to "Make a daily report on blanks furnished and send to the State Highway Department at Lansing at least every other day." This is the most satisfactory method of keeping a check on the men in the field. These blanks are so made that we can use them in our cost system of which I shall speak later.

The most perplexing question concerning alinement seemed to be the locating of lost corners. There are some lost corners which, to relocate, would cost more than the survey. We think it advisable to omit the location of such corners and to tie the survey to the next nearest corner. In general we do not relocate government corners unless the work can be easily done, but tie the survey to the next nearest section or quarter corner. This will sometimes entail a mile or more of chaining. The last sentence in Section 1 of the instructions, which states that "A monument shall be placed at the beginning and end of each survey and witnessed when such beginning or end does not coincide with a government corner," will correct the idea that was prevalent that such a beginning or ending needed no other marking than the first or last stake of the survey.

Section 4 states that "A curve shall be run in all cases where the deflection of the center line is 5° or over. In case of a 90° deflection the curve shall have a minimum radius of 66.0 ft. All curves shall be railroad circular curves without easement and shall be hubbed on the center line at each chord point in addition to the side hubs. Use 100 ft. chords up to 8° curves, 50 ft. chords 8° to 16° curves, 25 ft. chords on 16° to 32° curves and 10 ft. chords on all curves of 32° and over."

We had considerable difficulty with curves in the notes. Either the curves were not run in the field, or the chords were of such length as to divide the central angle into a number of equal parts or the curve data was so mixed up with topography notes that we, in the office, were unable to tell what was done in the field.

Section 5 instructs instrument men to "Note compass bearings of all lines and give transit angle for all changes in course. Do not use the difference between compass bearings for the transit angle, but read the verniers." We use the compass only to keep our sense of direction and as a crude check on the work.

In level work the check levels were omitted, benches were often "side shot benches" and too few notes were taken in cross sectioning. In a large portion of the work we do not set slope stakes and the cross sections are for the purpose of laying grades. We plot up the section at each station or plus and figure a grade that will give a balance between cut and fill provided we can do this without sacrificing other considerations which are often more important.

We require that at least five shots shall be taken at each section and enough sections taken to show the continuously changing shape of the road.

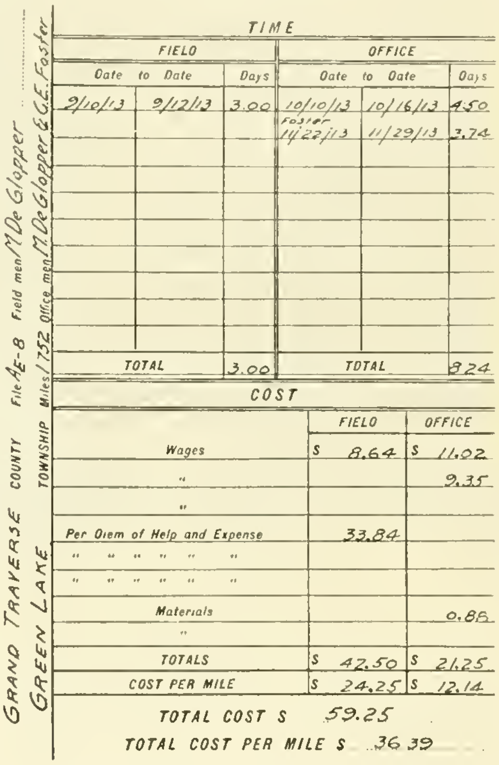
Under "Levels," Section 3, paragraph 2, the instructions require that "The five principal shots for each cross section shall be, one on each hub, one on each ditch line 14 ft. from the center line and one on the center line of the survey. Other shots with distance out from the center line shall be taken in sufficient number that a true cross section may be platted and end area may be calculated therefrom."

The remainder of the instructions under "Levels" deals with the detail instructions for level work and insists that all bench marks be included in the level circuit and all work be checked and proved before leaving the field.

In drainage we have had considerable difficulty. We do not go into this in as much detail as some other states have done. We do not require a plane table survey for drainage but the various areas draining toward the road are estimated, flood water elevations are noted, existing culverts and bridges, and possible new bridge and culvert sites are examined, cross road drainage is inspected and outlet ditches where necessary are surveyed. The drainage is checked by the man who makes the inspection for grade. Drainage is the vital question of road building and too much attention, in my opinion, cannot be paid to it.

Under "Topography," Section 1 reads: "The topography notes shall show the features of the adjacent territory (for a distance of at least 200 ft. from the center line) which may affect the design of the highway. The objects shall be platted by a system of right angle offsets."

Section 2 states, "The topography shall include the location of property line fences, pole lines, railway tracks, buildings, drives, intersecting roads, streams, ditches, trees, holes, swamps, sidewalks, crosswalks, retaining walls, guard rails and any other or unusual features. The names of property owners shall be recorded."



GRAND TRAVERSE COUNTY File AE-8 Field men M De Glopper
GREEN LAKE TOWNSHIP Miles 1752 Office men M. De Glopper & G.E. Foster

TIME

FIELD			OFFICE		
Date	to Date	Days	Date	to Date	Days
2/10/13	9/12/13	3.00	10/10/13	10/16/13	4.50
			Foster 11/22/13	11/29/13	3.74
TOTAL		3.00	TOTAL		8.24

COST

	FIELD	OFFICE
Wages	$ 8.64	$ 11.02
"		9.35
"		
Per Diem of Help and Expense	33.84	
" " " " " "		
" " " " " "		
Materials		0.88
"		
TOTALS	$ 42.50	$ 21.25
COST PER MILE	$ 24.25	$ 12.14

TOTAL COST $ 59.25
TOTAL COST PER MILE $ 36.39

Fig. 1. Card Used in Chief Engineer's Office, Michigan Highway Department, for Filing Costs of Highway Surveys and Plans.

Road Foundations and Materials.—Under "Soil Foundations," we desire to know the width, depth and character of the present road surface and the nature of the foundation material. This is highly important to an intelligent improvement of the road. Detailed instructions are given for this part of the work.

Under "Materials" we desire to know the quantity of available surfacing material, its location and cost.

Inasmuch as we keep the notes for alinement, topography, soil and foundation in the same field book it makes this book rather crowded. However, if the field man is neat and accurate in plotting his notes it is not difficult to get on one page the notes for twelve stations.

The level note book contains the level notes, check levels and a sketch of the cross section as often as may be necessary to give a clear understanding of the appearance of the ground to a person who has never seen it.

One of the questions most often asked is, "What is the cost of the surveys and plans?" In order to answer this question intelligently a three card system of cost keeping was devised. In the files all Trunk Line work is indexed by townships and the work in one township is kept separate from that in another.

A field report sheet was made out which gives the location of the work, the wages and expenses of the party, the weather conditions, the amount of work accomplished and the address of the party. The reverse side of the sheet is used for correspondence. One report is made out for each day and these are sent into the office as often as every other day.

For the office each draftsman has a weekly file card with seven columns headed in order, "County," "Township," "Division," "Material," "Day," "Hours," "Description." These are filled out as the work is done and turned in every Saturday. This card gives the time spent on the various jobs by the draftsman, together with the date on which the work was done. The cost card shown by Fig. 1 is arranged with the name of the county and township, the file number, the miles covered by the survey and the names of the men doing the work. Space is provided for the time spent in the field and office, and for the cost in field and office. The sum of the two totals gives the total cost of the survey, and the sum of the per mile cost for field and the per mile cost for office gives the total per mile cost, the figure which answers the above question.

On the reverse side of the card is a township plat on which the road is plotted. Below is a space for remarks in which to note items of interest concerning the work.

We find so far that the cost is running consistently with the conditions met. The cost of surveys, that is, reconnaissance and location and making plans for the same, have averaged about $50 per mile. Although there were a few miles which cost more than we expected the average is reasonable. The men engaged in the work were without previous experience in highway work. They had no previous methods which we had to override and they learned how to do the work as we want it done. I believe this is the best method. With these experienced men and a better organization for next season, we hope to lower the average per mile cost for this past season and I see no valid reason why it cannot be done.

The force now includes Mr. F. F. Rogers, State Highway Commissioner, Mr. L. C. Smith, Deputy Highway Commissioner, a bridge engineer, a chief draftsman, 15 engineers and draftsmen, a bookkeeper, three stenographers and a blueprint boy.

A Combination Liquid Asphalt and Oil Heater, Atomizing Pressure Distributor and Bituminous Road Patcher.

(Staff Article.)

In an article on the construction of the oil macadam road system of Los Angeles County, Calif., published in Engineering and Contracting May 24, 1911, a description was given of the trailer type of Ward's Liquid Asphalt and Oil Atomizer. The distributor is now made in two types, (1) a combination distributor mounted at the rear of a 750-gal. oil tank on a steel truck, and (2) a trailing atomizer as mentioned above. Both machines are arranged to be used as oil heaters, pressure distributors and for patching and flushing purposes. The trailing style of machine is of especial advantage where more than one tank wagon is required on the work, and the tank mounted or combination type is probably the better type for town and city work where short turns must be made and where the haul from the oil supply is not too great.

The combination machine is shown by Fig. 1. It consists of a 750-gal. oil tank mounted on a steel truck with a fire box beneath the tank for heating the liquid asphalt, oil or tar to the desired temperature. A Gould's geared rotary pump connected to a 5-HP. hopper cooled distillate or gasoline engine by a friction clutch cut-off coupling forces the oil under a pressure of 50 lbs. per square inch hrough a number of nozzles mounted on the discharge piping at the rear of the truck.

The nozzle boot of the machine is so arranged by a rack and pinion that it can be shifted laterally, making it possible for the operator to meet the edge of a parallel application without overlapping here and skipping there. The nozzles are connected by a detachable link to a lever rod, permitting the interchange from one to all nozzles or vice versa.

In a tee at the right end of the boot is a connection for attaching a hose for the patcher. The patching device is a 3 ft. length of one inch pipe fitted with handles and a pair of nozzles at the lower end, the nozzles are controlled by twisting one of the handles. A length of flexible metallic hose connects the patcher to a valve at the end of the boot. This equipment makes it possible to start out in the morning with a tank full of hot oil and keep going all day, accomplishing a great deal of work with a minimum amount of labor, trouble and lost time.

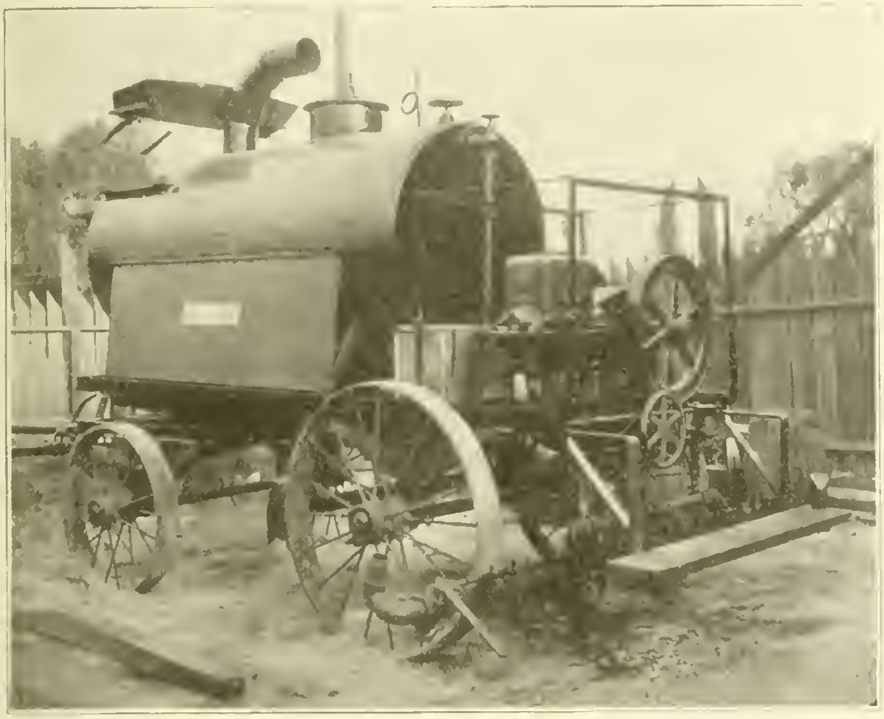


Fig. 1. Combination Road Oil Heating, Distributing and Patching Machine.

At the left end of the boot is a 2-in. valve, controlled from the driver's seat by rod and lever for operating a flushing nozzle which will deliver 65 gals. of water per minute at 30 lbs. pressure, for use in flushing streets. The machine may also be used as a sprinkler by operating spray nozzles.

The piping system is arranged with a 1½-in. by-pass, connected into a tee at each end of the boot back to the top of the tank. This by-pass line affords a means of circulating oil throughout the entire system for warming pipes and more quickly heating the oil uniformly to the desired temperature. Oil suction and discharge lines are 2 in. A flexible joint is provided in the discharge and by-pass lines, to allow shifting of the boot, by fitting a 2½-in. section of pipe with a 2½ x 3-in. reducer, inserting the end of a 2-in. pipe and packing by screwing a 2 x 3-in. bushing into the reducer after the packing has been placed, the inside of bushing being turned out smooth. The joint in the by-pass is formed in the same way, using 1½-in. and 2-in. pipe. The advantages claimed by the manufacturers for the Ward combination machine are as follows: (1) The machine is designed for constructing oil macadam, top dressing macadam, gravel and dirt roads with liquid asphalt, heavy asphaltic oil or any of the road oils or tar products; (2) the machine can also be used for spraying streets with light oils or emulsions for laying dust; (3) it will distribute from 1/12 gal. to 1 gal. of oil per square yard of road surface; (4) full control of speed, pressure and amount of material distributed is at the command of the operator; (5) requires the use of less oil on account of the effect of atomizing and even distribution of oil; (6) allows the use of a heavier oil containing a larger percentage of asphalt making a better and more durable road; and (7) the machines are built of the best grades of machinery and material suitable for the service.

Method and Cost of Constructing Slag Roads Near Pascagoula, Miss.

Contributed by Charles E. Chidsey, Road Commissioner, District No. 3, Jackson County, Mississippi.

The first task of the Commission was to organize a working force. Mr. W. A. Hooks of East Side, Miss., who had had considerable experience in working convicts on public roads, was selected for foreman over a force of six men and a cook. The foreman's salary was $60 per month and laborers were paid $1.50 per day. An old railroad box car was mounted on wheels and fitted up with bunks, cook stove and utensils for a camp which could be easily hauled from place to place by a team of mules. The men were charged 50 cts. per day for board and lodging at the camp. An 8-ton horse-drawn roller and a Galion Ideal Giant grader which were already owned by the county were the only machines furnished at that time.

The first road selected for improvement ran for nearly four miles through a flat level country scantily clad with pines and known locally as "pine meadows." These pine meadows are in a rainy season so wet as to be almost impassable and the old road was for long stretches a mere bog. The first task was to set an engineer at work to lay out a plan of drainage. Ditches were cut on either side parallel with the road, and one large lateral ditch 3 ft. wide and 1 ft. deep was extended for nearly a half mile to an outlet north of the meadows which effectually conveyed the water away from the parallel ditches at the side of the road.

When the ditching had been completed the grader was put at work shaping up the road. When the grading was finished the road was rolled with the 8-ton roller and was then ready for traffic. The Commission was then confronted with the problem of finding a suitable surfacing for the road and the majority were in favor of slag but as the quoted freight rates from the furnace to Pascagoula were so high and no reduction could be had, the Commission determined to cover the road with oyster shells, a material that has from time immemorial been used on the Gulf Coast for road beds. The shells were brought by barges via Mississippi Sound and up the Pascagoula River from Biloxi to Pascagoula from whence they were hauled in wagons to the point where they were to be used. The hauling was done by contract, the contractor spreading the shells upon the surface of the road. The shells cost 22.5 cts. per barrel put upon the road and the cost per mile of this road including grading was $1,400.

When all of this road had been completed except about ¼ mile events occurred that changed the whole of the Commissioners' plans and one that may mark an era in the history of road construction, in this locality at least. The Commissioners had other shell roads under construction, the details of which need not be mentioned, and just as the construction of the road described above was nearing Laine, the supply of oyster shells gave out. Thanks to the efforts of The Birmingham Slag Company, Birmingham, Ala., a rate on slag was given by the railroads that made it possible to put slag on the roads at a reasonable cost. From the depot at Laine there was a dirt road 22 ft. wide which extended one mile due north to the Pulp and Paper Mill. This road had been constructed by the Board of Supervisors and it was included in the 3rd district when the Commission for Road District No. 3 was organized. It ran through a stretch of country that easily admitted of drainage and was paralleled by a side track of the L. & N. R. R., which afforded the ready discharge of material brought by the railroad. The Commission determined to use this road as a test of slag as a material for road-building.

The surface of the road was first hollowed out to a depth of 4 to 6 ins., according to the foundation found and then filled with slag. When covered with slag to this depth the slag was crushed down into the foundation by running the 8-ton roller over it until a smooth surface was obtained. Then the soil was raked back into place and the road again rolled. At first the road was somewhat lumpy but it soon packed down hard and smooth. In fact it is as hard and almost as smooth as a concrete pavement. While waiting on this experiment the Commission continued its work of constructing other dirt and shell roads, but when the value of the slag road was seen they agreed to cover all the main highways in the future with slag.

The work of constructing a highway from Laine to Orange Grove, a distance of nearly five miles and which makes a link in the proposed speedway from Mobile to Biloxi, was at once begun. Orange Grove is on the L. & N. R. R. and convenient for the delivery of the slag. The work of re-constructing "The Old Mobile Telegraph Road," which was built before the Civil War to connect Pascagoula with Mobile, was also started and this road was also selected for "slagging." The details of the work on these two roads are the same as described for the experimental slag road, as they go through the same flat country.

These roads have been completed some months and their condition justifies the expense and labor spent upon them. Only a few days ago the writer and the foreman, Mr. W. A. Hooks, inspected all the slag roads and they have packed down as hard and smooth as the road from Laine to the Pulp and Paper Mill. It is, however, to be observed that where the road goes through sandy soil and sand has been used for a covering on the slag small ruts from 1 to 3 ins. deep and 2 ins. in width appear on the crown of the road where they have been cut by the tires of passing vehicles, while wherever the roads have gone through clay or loam there are no ruts, the road being as hard and almost as smooth as concrete. Broad tired vehicles such as automobiles press down these ruts and fill them up making a smooth surface.

AMOUNT AND KIND OF SLAG USED.

It should be stated that wherever the road was cut through a foundation of clay, the slag was only put on to a depth of 2.5 ins., while on sandy foundations the average was 6 ins., the clay foundation causing a saving of 58 per cent of the material. The slag cost $2.00 to $2.25 per ton placed upon the roads according to location. It should also be stated that the slag that has been well crushed before shipment is better for roads than the large lumps. The large lumps must be broken by a sledge or axe causing much loss of time and labor. Often the large lumps of slag cannot easily be broken, thus making a waste. The small size slag is the more easily mashed down into the foundation and makes a more compact mass.

In November, 1913, the Commission had to reorganize its working force and begin anew owing to the purchase of new machinery. A 40-h. p. Flour City Gasoline Tractor sold by the Galion Iron Works, was selected for hauling and power purposes. This machine with a force of five men will in good weather grade 25 miles of road per month. This does not include the cutting out of right of way. Two graders and a scarifier were used with it. Under the contract system the Commission had let out contracts for roads that cost from $75 to $250 per mile, while with this gasoline tractor it can construct dirt roads at a cost of $25 to $35 per mile.

The tractor has also been used when the weather was good, in clearing right of way, pulling up trees and stumps—a work that was done previously only by hand or blasting with dynamite. During wet weather when the soil is so soft that it will not permit of the weight of the heavy tractor the men are used for blowing up stumps and digging ditches, but no ditches are dug or right of way cleared out until the lines have been carefully laid down by an engineer. Since the gasoline tractor was placed in service the foreman is paid $75 per month, the tractor engineer $75, and five men are employed at $1.50 per day. The tractor consumes about ten gallons of gasoline per day. The Commission has disposed of all its mules except two which are used for hauling from places where the tractor cannot well go and are used in grading where the soil is too wet to admit of the heavy tractor. The machinery in use is the 8-ton roller, 40-h. p. tractor, two graders and one scarifier.

The writer has from time to time heard and read the statement "that in a flat country the services of an engineer are not needed to secure the proper levels for drainage." No statement could be more misleading. In stretches of level country the fall perhaps will be only a few inches in several hundred yards and any attempt to dig a ditch and get drainage without the proper use of engineer's instruments is an idle waste of money and labor.

During the year 1913 the Commission constructed about 15 miles of shell road, 7 miles of slag road and 140 miles of dirt road. The roads constructed with shells cost on an average about $1,400 per mile and the slag roads cost $1,600 per mile. However slag roads are the cheapest as the life of a shell road is from three to five years, while the life of a slag road is practically unlimited. Oyster shells or clam shells under the continual grind of the wheels of passing vehicles become an impalpable dust which blows with every wind, and washes away in every rain soon leaving deep ruts in the road. Again in dry weather a swift moving vehicle like a motor car raises a cloud of dust from the shells that finds its way into the mouth and nostrils of every person along the road. The shell road constructed in 1912 and 1913 now needs repairs and it is the purpose of the Commissioners to make these repairs with slag, as the small piece of road near Laine that is a mixture of slag and shell is a most ideal road. The slag and shell make a compound that is very good and were it not for the expense would be the most desirable material. The slag never grinds down into powder, it neither blows or washes away nor does it give off any annoying dust. As time goes by it becomes harder and smoother. A shell road is a thing of beauty and adds much to the attractiveness of the landscape and were it not for the faults we have named would be a joy forever.

Now the conclusions to be drawn from the facts I have stated as we have learned by experience in making good roads are:

(a) All road work should commence with a system of good drainage. Good drainage is the sine qua non of good roads.

(b) That slag mixed with clay, loam or shells makes the best roads and that while sandy soil is good it is not the best material for a foundation or for covering the slag.

(c) While oyster or clam shells make beautiful roads they are more expensive because less lasting than slag.

(d) In using slag small pieces of slag are preferable and a roller weighing at least 8 tons should be used for crushing. One of lighter weight will not do.

(e) That by using a gasoline tractor the cost of grading roads is reduced from 60 to 75 per cent.

(f) It costs less to the Commission to construct its own roads under an efficient foreman than under the contract plan.

(g) When you have a good working organization it is better to keep the men at work when the machinery cannot be used as otherwise their idleness weakens the morale of the force.

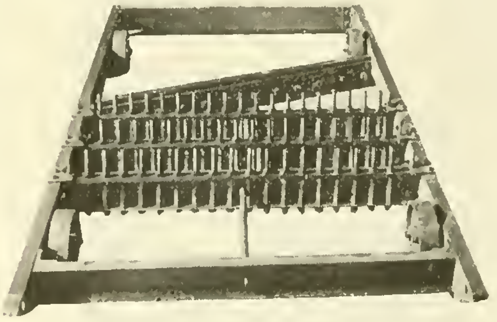


Fig. 1. End View of the Bituminous Road Planer.

(h) In laying out roads and ditches a competent engineer ought always be employed to get levels. Any other method is false economy.

A Planer for Smoothing Bituminous Road Surfaces.

(Staff Article.)

An inexpensive bituminous road planer which has been used in a number of places including the road departments of Massachusetts and Connecticut is illustrated by the accompanying engraving. Aside from the illustrations, the general construction needs very little explanation. One fact to be noted, however, is that the wheels extend just far enough below the runner bottoms to prevent friction, and that they can be shifted to a lower set of holes to raise the runners well off the ground when the scraper has to be hauled to a new place of operation.

The smoothing devices consist of three rows of cutting teeth held by beams extending straight across the machine and of diagonally set planer blocks. The cutting teeth are ¾-in. square tool steel rounded and threaded in the top portion to permit raising and lowering by double vents. These teeth are so set diagonally in the holders that a sharp edge is forward. The teeth are removable and so is the planer block, which is made in three interchangeable parts. Since the center part of the planer wears fastest this interchangeability permits the periodical shifting of the side parts to the center and the center parts to either side, thus providing even wear of the blade across the planer. The blade sections are also double edged and can be inverted when one edge is worn beyond further use.

In operation the planer is hauled by team or mechanical power. The forward row of teeth maks a thin cut, the middle row cuts deeper and the rear row cuts to a depth flush with the edge of the plane block. The plane removes irregularities left by the teeth, smooths the surface and scrapes the chippings into depressions or to one side through a notch at *A* in the runner. When it is desired to scrape the chippings toward the center the planer is hauled in the direction of the traffic; to scrape the chippings toward the road sides the planer is hauled in opposite directions to the traffic. The machine is weighted down when necessary.

The planer described costs $200 at the factory and is made by the Headly Good Roads Co., Philadelphia, Pa.

Object-Lesson and Experimental Roads.—Forty-one object-lesson roads were constructed by representatives of the U. S. Office of Public Roads under the direct supervision of Benjamin F. Heidel during the fiscal year 1912-1913, according to the annual report recently issued by the Office of Public Roads. The new roads may be classified as follows: Two bituminous-macadam; 7 plain macadam; 4 gravel; 1 gravel macadam; 14 sand-clay; 1 sand-gumbo; 3 shell; and 9 earth. The total cost of the work, exclusive of salaries and expenses of engineers furnished by the Office of Public Roads, was $92,151.

Three experimental roads were constructed which consisted of work along various lines, as follows: Bituminous-concrete, bituminous-surfaced concrete, oil-cement concrete, cement concrete, brick, surface treatments, bituminous-macadam, shell and macadam. Table I is given to show the number of square yards of each type of object-lesson roadway constructed during the fiscal years 1905-1913 and experimental roads for 1913, inclusive.

The Engineers' Club of Cincinnati at its regular monthly meeting on March 19, was addressed by one of its own members, Mr. F. W. Willey, Electrical Engineer with the Triumph Electric Company of Cincinnati, who spoke on the subject of "Types and Ratings of Commercial Electrical Machinery," and illustrated his address with stereopticon slides.

TABLE I.—AREA IN SQUARE YARDS OF OBJECT-LESSON ROADS CONSTRUCTED DURING 1905-1913, INCLUSIVE.

Material—	1905.	1906.	1907.	1908.	1909.	1910.	1911.	1912.	1913.[1]
Brick	......	......	......	......	......	526	......	......	2,055
Concrete	......	......	......	......	......	1,004	......	......	3,013
Oil-cement concrete	......	......	......	......	......	......	1,917	......	1,744
Bituminous concrete	......	......	......	......	......	......	......	......	2,898
Bituminous-surfaced concrete	......	......	......	......	......	......	......	......	4,178
Bituminous macadam	......	......	......	......	......	45,832	41,551	31,453	16,040
Surface treatment	......	......	......	......	......	......	......	......	6,386
Macadam	44,944	51,246	76,376	72,587	96,107	50,333	11,330	14,806	57,131
Asphalt-slag	......	......	......	......	......	......	578	......	......
Oil asphalt gravel	......	......	......	......	......	......	900	......	......
Oil-gravel	......	......	......	......	......	4,819	9,774	......	......
Gravel-macadam	......	......	......	......	......	......	......	......	2,607
Gravel	8,804	4,197	11,722	4,608	65,793	71,376	59,942	13,057	63,730
Slag	......	......	......	......	......	4,610	......	......	......
Sand-clay	19,178	19,443	85,571	12,634	205,032	177,960	218,177	103,876	128,496
Sand-gumbo	......	......	......	......	......	......	......	......	5,337
Burnt clay	100	......	......	3,392	2,041	......	......	......	......
Shell	5,877	933	......	14,020	1,630	......	......	......	43,717
Earth	......	12,132	27,042	85,967	319,456	651,109	140,933	556,663	162,696
Total	79,203	87,951	200,711	223,208	690,059	1,007,569	485,102	722,855	500,028

[1] Includes experimental roads.

Railways

Construction of the Watauga & Yadkin Valley R. R., With Details of Methods and Costs of Earth and Rock Excavation.

Contributed by H. C. Landon, General Manager, Watauga & Yadkin River R. R. Co., North Wilkesboro, N. C.

LOCATION.

The Watauga & Yadkin River R. R. is a new standard-gage line under construction from North Wilkesboro, N. C., where connection is made with the Southern Ry. to Boone, N. C., a distance of 52 miles. An extension is proposed from Boone to Butler, Tenn., where connection will be made with the Virginia & Southwestern Ry., giving a through route over the Blue Ridge and Stone Mountains. The importance and advantage of the line to the territory to be served may be realized when it is stated that the nearest through line crossing these mountains north of the proposed line is 50 miles away, and the nearest line in the other direction is 30 miles south. The new railroad will shorten the haul on coal from the mining regions to the Atlantic coast by nearly 150 miles. Branch lines of the W. & Y. V. R. R. are planned from Elkville, N. C., to Lenoir, N. C.; from Bamboo, N. C., to Blowing Rock, N. C.; and from Bamboo to Jefferson, N. C. The final location of the main line was commenced at North Wilkesboro in June, 1912, but construction did not get under way until about Aug. 1, 1912. The construction of 21 miles of line between North Wilkesboro and Grandin was completed June 1, 1913, and 12 additional miles of the main line between Elkville and Darby, which were graded and nearly ready for the rails at that time will soon be placed in operation. On July 1, 1913, the railroad company began operating the 21 miles to Grandin. The portion between Elkville and Grandin of the branch to Lenoir was built before completing the main line in order to handle a large amount of freight offered by the Grandin Lumber Co., which has a large sawmill and lumber industry. Figure 1 is a map of the line and its connections.

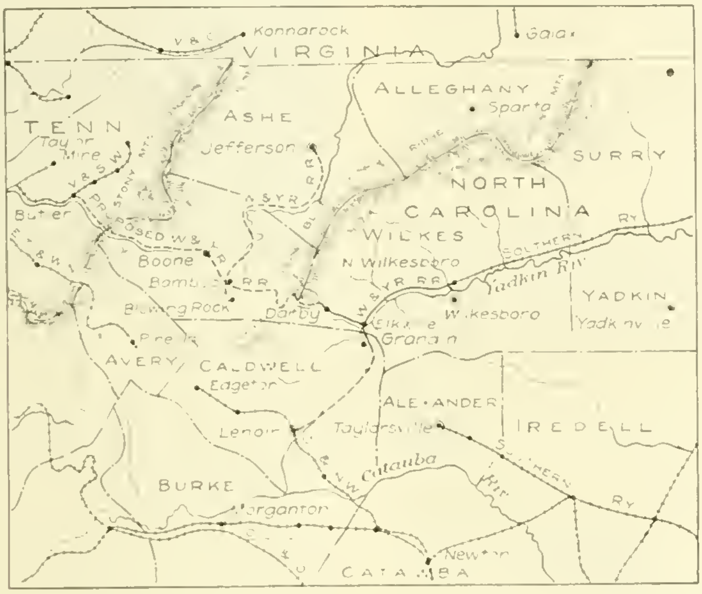


Fig. 1. Map Showing Location of the Watauga & Yadkin Valley R. R.

GRADES AND CURVES.

Although the road is generally constructed through rough country, necessitating the use of one tunnel in the section of the line already completed, the grades are so planned as to provide for economical handling of the expected heavy freight traffic of mica, iron ore and lumber. The maximum rate of grade allowed on mountain sections is 2.2 per cent compensated. At all other points the maximum gradient does not exceed the maximum gradient of the streams by more than 25 per cent, unless the railroad grades are short and no lighter grade could be secured. In no case are opposing grades allowed against a down-hill haul. Vertical curves are used at grade intersections where necessary. Grades are compensated for curvature at the rate of 0.04 ft. per degree per 100 ft. on curves up to 8°, and at the rate of 0.03 ft. on curves over 8°. Although a large proportion of the curves on the constructed line are less than 8°, topographical and economical reasons required the occasional use of sharper curves, which are to be revised when conditions warrant the expense. Transition curves are used on all curves of 3° or over.

Grading.

The methods followed in construction were rather unusual, as the company was its own contractor. In other words, the grading was done largely by its own forces under the direction of the Chief Engineer. One small six-mile section was constructed by contract. It was believed by the organization that it could build its line with greater speed, than by contract, and with less annoyance, as it was known that numerous changes of the alinement would necessarily be made.

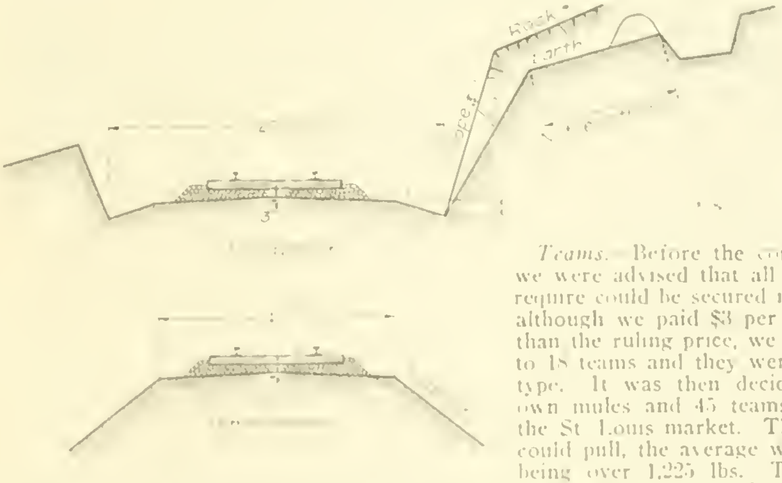


Fig. 2. Standard Roadbed Cross Sections Watauga & Yadkin Valley R. R.

At present, however, as the railroad company has some mileage in operation and a considerable mileage graded, and because its officers will have other duties, it is believed that the greater portion of the balance of the line will be built by contract. It is also believed, however, that by the methods followed and the careful study of the work done, that more work was accomplished in the small space of time available than otherwise would have been accomplished. First, no time was lost in starting the work, the moment it was determined that the section of line was definitely located; second, there was no waiting upon contractors to look over the work and bid upon the same, as all arrangements had been made and prices secured so that all equipment could be ordered by wire.

The profile of the first 30 miles of projected line, indicated generally light work. There were some heavy cuts and fills, but it was not practicable, on account of poor roads and lack of bridges to entertain any steam shovel proposition, and generally the cuts and fills were too light to make steam shovel operation economical. Labor was scarce, and at a premium everywhere. The plan was to do the work with the aid of teams, machines and powder as far as practicable.

Equipment.—After a careful study of the projected profile, it was decided to purchase the following equipment:

12 1½ cu. yd. Troy wagons at $112.50	$1,350.00
24 drag scrapers at $5.56	133.44
36 No. 2½ wheel scrapers at $36.75	1,323.00
1 elevating grader at $920.00	920.00
4 2,500 lb. wagons at $55.00	220.00
8 16 ft. by 24 ft. tents at $38.63	309.04
2 32½ ft. by 65 ft. mule tents at $149.30	298.60
2 Ingersoll rock drills at $312.50	625.00
1 16 hp. boiler on wheels, 2d hand, at $300	300.00
10 1 yd. dump carts with harness at $46	460.00
4 2 yd. dump cars with harness at $30	120.00
100 steel wheel barrows, 3 and 4 cu. ft. at $3.00	300.00
12 doz. round Pt D handle shovels at $5.25	63.00
4 blacksmith outfits, including a forge, anvil and other tools, at $40.00	160.00
12 doz. picks with handles at $4.00	48.00
Total	$6,630.08

The dump wagons and grader were only used about two months and did fair work in the territory where they were used. They were not used for a longer period on account of inability to get sufficient mules and teams to operate them.

Teams.—Before the company started work, we were advised that all the teams we would require could be secured in the community, but although we paid $3 per day or 50 cts. more than the ruling price, we could only secure 15 to 18 teams and they were not all of the best type. It was then decided to purchase our own mules and 45 teams were purchased in the St. Louis market. These were mules that could pull, the average weight of the animals being over 1,225 lbs. These teams were in almost continuous daily service from Aug. 1, 1912, to June, 1913. Only two mules were lost and it is estimated that there was not over 5 per cent lost time for the mules in service. Our cost of feeding the mules averaged 95 cts. per team per day. For all camps hay averaged $25 per ton and oats 57½ cts. per bushel delivered at the camps.

On June 1, 1913, the work was shut down for short time. At that time had our mules gone on the market they would have brought more than they cost, as their condition was excellent, due to their excellent care. It was the theory of the company that only from well fed and well kept teams could satisfactory efficiency be secured. The teams were taken care of by a competent stable boss. A supply of necessary remedies and an emergency case were kept at each camp. The fact that there was not over 5 per cent lost time for the teams explains the low price and rapid progress.

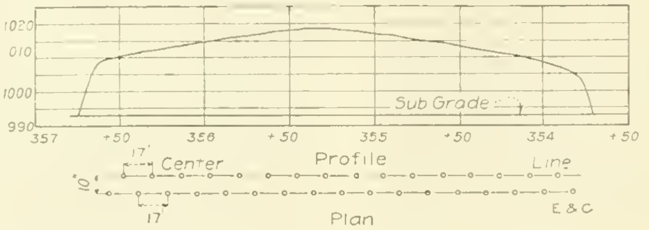


Fig. 3. Plan of Blast Holes in Cut Sta. 353 to Sta. 357, Watauga & Yadkin Valley R. R.

EARTH EXCAVATION.

Organization of Forces.—It was realized that in order to secure efficiency, the organization of the various forces should be fixed. These forces were called "Standard" and only varied when it was shown that the nature of the work demanded it. Thus the Standard wheel scraper force was as follows for hauls not exceeding 300 ft.: 6 wheel scrapers with teams and drivers, 2 teams to plows, 1 snatch team, 1 man dumping, 1 loader, 1 wheeler, 1 waterboy, when required, and 1 foreman.

When the haul increased the number of wheel scrapers was increased, in order to keep the snatch team and other laborers busy. This was very closely watched by the foremen of the various gangs, in order to keep up their records, as every dumper was supplied with a counter and the day's work reported. In this way, a very close estimate could be made of the yardage moved.

The "Standard" drag scraper force consisted of 6 scrapers, with teams and drivers, 2 teams to plow, 1 dumper, 1 loader, who acted as foreman, and 1 waterboy.

The drag scraper work and the wheel scraper work were watched with great care to determine the economical haul. The drag scraper is very efficient and very useful for very short hauls only. Observation of the various hauls up to 200 ft., fully demonstrated the fact that for a distance of over 100 ft. the drag scraper was an expensive implement. Under 100 ft. it will do efficient work. Wheel scrapers ordinarily can be used where the drags can, but had the advantage in making about the same speed with about five times the load. As a general proposition, only a few drags should be used on work of this character. Their advantage is in their cheapness and for a small amount of work with short hauls the drag scraper might be desirable equipment. A gang of wheelbarrow men properly handled will do work about as cheap as a drag and in some instances at less expense.

Cost of Drag Scraper Excavation.—Assuming the haul for a drag scraper to be 100 ft., a lively mule team to a scraper will not make over 1.3 miles per hour on account of the frequent turns and loading, or about 6,900 ft. per hour. This will be at the rate of 3.45 cu. yds. per hour or 34½ cu. yds. per 10-hour day per team. With a "Standard" drag scraper force and teams at $3 per day, 8 drags will handle 27.6 cu. yds. each per 10-hour day, at a total labor cost of $37.50, or nearly 14 cts. per cubic yard. With a haul of from 50 to 75 ft. the cost will not exceed 12 cts. per cubic yard. Seventy-five ft., therefore, should be the maximum haul with drag scrapers. Six drag scrapers with the shorter haul were therefore established as the maximum to use with the minimum haul.

Actual observation on a 110-ft. haul with country teams, indicated that under the best conditions only 25½ trips were made per hour, or a speed of 5,000 ft. per hour. The company teams which were all well fed Missouri mules, made as high as 7,200 ft. per hour with drag scrapers on a haul of 150 ft.

These results were obtained under the best possible conditions where the dumper man counted and reported every load and in addition the teams were under my personal observation.

A few drag scrapers on every job of similar character are a good investment, but their number and use should be limited. An injudicious foreman will often use them at the company's expense.

Our standard forces were modified as the hauls increased, the number of wheelers increased to eight, and, possibly, with very long haul ten or even twelve wheelers could be used to advantage.

Cost of Wheel Scraper Excavation.—In only one instance did the haul with wheelers much exceed 600 ft. and in this instance the haul averaged 1,350 ft.; ten wheelers only were available, but we were able to handle 225 cu. yds. at a cost of 23½ cts. per yard, figuring teams at $3.50 per 10-hour day, although all the teams which we used actually only cost us approximately $3 per day.

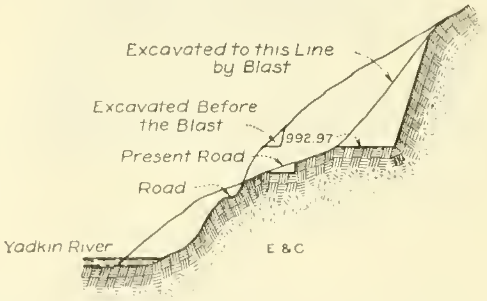


Fig. 4. Cross Section of Cut at Sta. 355+34, W. & Y. V. R. R., Showing Original Ground Surface, Section After Blasting and Completed Section.

With a haul of 415 ft. a careful timing of the teams indicated that they were making four trips in 20 mins. The average of twelve trips per hour was made for the entire day. The wheelers were loaded to their capacity, and therefore, an average of nearly 60 cu. yds. per wheeler was secured. The wheeler force using only six wheelers cost $30 per day. The labor cost in this case did not much exceed 10 cts. per yard.

Wheelbarrow Excavation.—The wheelbarrow when properly used is a most useful, necessary, convenient and economical tool. Three types of barrows were purchased. The ordinary railroad wooden barrow, the wooden frame contractors barrow, with steel tray, and the all steel wheelbarrow with one piece tubular bent handles. Barrows of 3 and 4 cu. ft. capacity were bought. The barrow holding 4 cu. ft. in general seemed to suit our work and could be handled about as easy as the barrow holding only 3 cu. ft. The ordinary wooden wheelbarrow gave very poor service. We had a few of the barrows with wooden frames go out of service, but the all steel wheelbarrows are practically as good as new after 8 months fairly good service. The barrows are painted when out of service any length of time.

For side-hill work and to open grade points the barrow is giving very efficient service. Observation on side-hill work with a gang of 25 men handled dirt at the rate of eight wheelbarrows per minute for an hour with a haul of 21 ft. This would mean that they moved over 500 cu. yds. in wheelbarrows holding 4 cu. ft. Good runways were always provided, so that the loads could be moved with the least possible waste of energy.

The gangs were placed in the hands of efficient foremen, who taught the men how to handle the most dirt with the least possible loss of time. It was endeavored at all times to have a standard gang of not less than 25 men under each foreman. The work varied as the conditions necessitated. In some cases much drilling was required, in others none at all.

Cost of Dump Car and Cart Excavation.—Four small dump cars with revolving bodies were found to be convenient and useful in short cuts and at the approaches to the one tunnel that has been built. These cars run on a track of 30-in. gage and had a capacity of 2 cu. yds. The cars were particularly useful in small cuts and where the haul was long. The revolving body would permit the car to be dumped in building the fill ahead of it and could be dumped on the side to widen the fill or waste the material. Light rails not being available these cars were run on a track made of 4 x 4 oak timbers. The wooden rails only required a few renewals during their six months of service.

Dump carts can only be used economically upon hauls about 100 ft. long, but two of the cars moved by mules would keep a gang of 10 to 12 shovelers continuously busy—where the haul was from 600 to 700 ft.

In one cut alone, it is estimated two of these cars handled 15,000 cu. yds. of earth and rock, with a maximum haul of 650 ft., at a cost not to exceed 20 cts. per cubic yard. The average gang, including drillers, was about 14 men and a foreman. This number of men loaded about 150 cu. yds. per day at a labor cost of about $25 per day. It took nearly three months to remove the cut.

While dump carts should be used for short hauls of from 100 to 125 ft., yet they have been used to advantage where the maximum haul was 250 ft., provided the roadway was kept in good order and several carts were used to keep a good sized gang moving. In one instance, six carts were used in completing a fill and did the work very rapidly where the haul was approximately 150 ft. Six carts and 30 laborers moved 325 cu. yds. per day, at an expense of approximately $47 or about 15 cts. per cubic yard. As a general proposition our cost of handling earth and rock with dump carts and men is about 26 cts. per cubic yard, exclusive of explosives.

Roadbed Cross-Section.—The roadbed cross-section is shown by Fig. 2. It is expected to ballast later with stone but the ballast section is not decided upon. On account of the character of the soil and the climate a slope of only ½ to 1 in earth cuts is necessary and advisable. I believe even steeper slopes would be advisable as experimental; ¼ to 1 slopes are standing very satisfactorily.

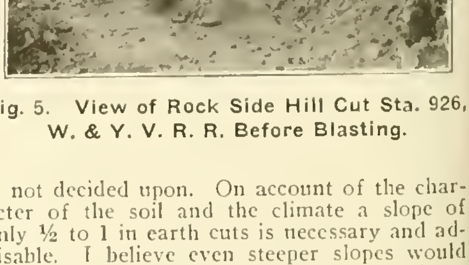


Fig. 5. View of Rock Side Hill Cut Sta. 926, W. & Y. V. R. R. Before Blasting.

ROCK EXCAVATION.

Methods of Using Explosives.—The line as located and the material afforded a splendid opportunity for the use of explosives. Much of the line was located in the side hills in locations where heavy blasts would be made without endangering life or property. The proper using of explosives, however, for moving earth has been given, as a general thing, very little attention. The experienced men that we employed had only used explosives to shatter and break up the rock or hard soil, so that it could be handled by either hand or steam shovels, and the old powder men at first tried to continue the use of that method. Much valuable time and good powder were wasted before we could so improve the use of powder that the blast would move the maximum amount of rock and earth out of or off from the railroad grade. Where dynamite is handled in large quantities, unless you fortunately have a general foreman who is familiar with the dangerous qualities of that explosive serious accidents are apt to happen if all precautions are not taken. No foreman or man should be allowed to handle explosives unless he fully understands the danger of careless handling of them. Rigid instructions should be issued and an inspector should be detailed to see that all instructions are carried out. A foreman permitted to handle dynamite should have had at least one year's experience in the handling of explosives. He should pass an examination as to his qualifications.

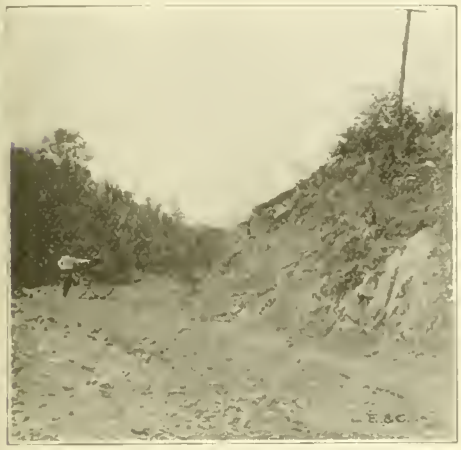


Fig. 6. View of Completed Cut Sta. 926, W. & Y. V. R. R.

One serious dynamite accident occurred due to disobedience of rules by a laborer. In this case the dynamite was being thawed by an open fire, which was strictly against the rules. The laborers of the night force about the tunnel were just finishing their midnight lunch at the fire when one of the sticks of dynamite caught fire and this communicated the fire to other thawing dynamite. All the laborers ran to a place of safety. One of the men, however, to show his disregard of burning dynamite and ostensibly to save the dynamite at the fire, ran back to the burning explosives and while reaching for the dynamite in a bending position, all the dynamite exploded, injuring the man slightly around the body but destroying both eyes. Regardless of all the facts, a jury has given him damages for a large sum.

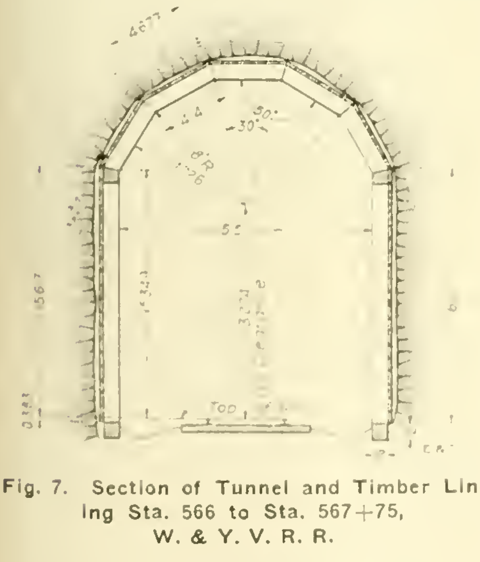


Fig. 7. Section of Tunnel and Timber Lining Sta. 566 to Sta. 567+75, W. & Y. V. R. R.

The first order for explosives was for a minimum carload of powder (400 kegs) and two tons of dynamite with sufficient fuse, caps, electric exploders and one No. 3 push down blasting machine. This was to be used on the first rock cut on the line where the powder was to be used only to break up the rock generally, so that it could be moved to the adjoining fill. The small amount was shot to waste. In the succeeding rock cuts, of which there was a number, it was the plan to move as much rock out of the cuts as possible by powder, as this was the cheapest possible plan of moving the material. Many of the cuts were side-hill in a thinly settled location as already stated.

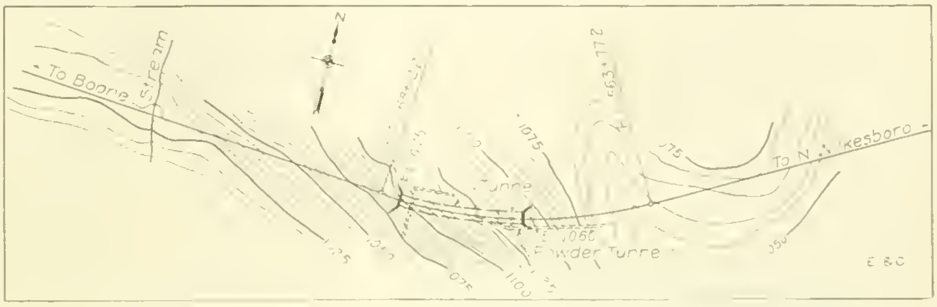


Fig. 8. Plan Showing Location of Tunnel and Powder Tunnels Used for Blasting Tunnel Approaches.

Methods of Spacing, Drilling and Loading Blast Holes.—It was difficult at first to get the desired results as all the old-time powder men rather believed in the single shot or two or three shots planned, the result being that a poor showing was made. I finally made arrangements with the general foreman to lay out or plan the blasts in the other cuts and for some time no heavy blasting was done except under personal direction. The cuts were usually of a length where 30 or less than 30 holes would cover the portion of the cut to be moved, so that a No. 3 blasting machine would explode the blast.

The holes were spaced on lines that have since entered into our general instructions for use of powder and other explosives. In general where the cut at the center line was over 4 ft., and the material earth or soft rock, the first line of holes was placed not more than 2 ft. above the center line. All holes were driven to a point 2 ft. below grade and usually about the same distance apart as the depth of the hole to grade, except when depth was greater than 15 ft. The maximum distance apart was about 15 ft. If the hillside was steep and the lower side of the road bed at grade, one set of holes was sufficient. If the cut under ordinary circumstances was a through cut with the depth of cut 2 ft. or more on the lower side, then a lower set of holes was drilled parallel to the first at the lower ditch line at points midway between the upper holes, so that there would be no question about moving the material out of way. This will not change materially the amount of powder used, as one yard of soft rock should be moved with about 2 lbs. of powder. The soft rock usually was a decomposed granite or a Carolina gneiss which was not hard to drill.

The hard rock encountered was usually a mica schist. This was very hard, and in these cases the upper line of holes was placed on the upper ditch line and a distance apart equal to the depth up to 10 ft. No holes were placed farther apart than this. The lower holes were placed on the lower ditch line at the same distance apart but intermediate to the upper line of holes.

The arrangement had to be modified according to the rock, as hard and soft rock would be found in the same cut that was to be demolished by a single blast. The arrangement of the strata changed our plans, although generally the strata we encountered was nearly vertical or leaned slightly to the northwest.

The general tendency was to use too much powder in the soft rock or earth and too little in the hard rock. A careful estimate was made of the quantity in the cut. If soft rock 2 lbs. or less were used per yard; if hard rock, 3 lbs. per yard or more. The general foreman in charge followed instructions carefully and used good judgment in his large blasts and usually wasted very little or no powder. After he became acquainted with the rock and the mistakes, mistakes were rare.

In all the smaller shots that are being made, the foremen are instructed to use powder judiciously, and they are getting good results with a minimum amount of powder. The holes are made on the center line to a point about 2 ft. below the grade line and are spaced a distance apart equal to the depth of the holes. It is found that the holes drilled on the center line and to this depth below the grade will ordinarily pull down the grade about the amount desired, and will not move the earth too far back of the slope line where soft rock is handled.

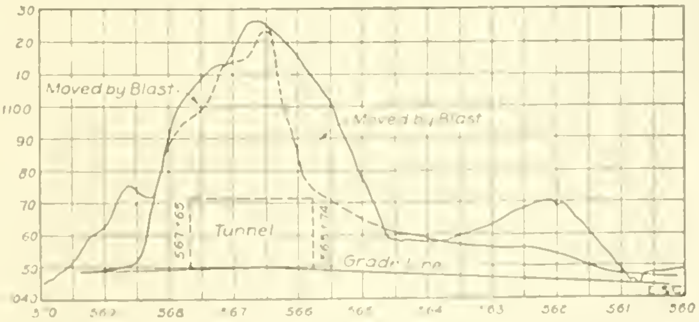


Fig. 9. Profile Showing Tunnel and Results of Blasting Approaches by Means of Powder Tunnels.

In all cases where there were over five holes, the battery was tested with a Rheostat and every fuse and finally the line was tested with a galvanometer, so that no misfires occurred. It is not rare that a faulty fuse is found, and when they do occur they can cause annoyance if they are used.

Drilling Outfit.—Steam drills are used with hard rock, while a large percentage of the other holes are put down by hand and churn drills. In many places churn drills were successful in soft rock. In all hard rock, steam drills are used when possible. The two drills used were the Ingersoll P-24 type with 3-in. cylinders 6½-in. stroke. One 12 HP. boiler supplied the two drills.

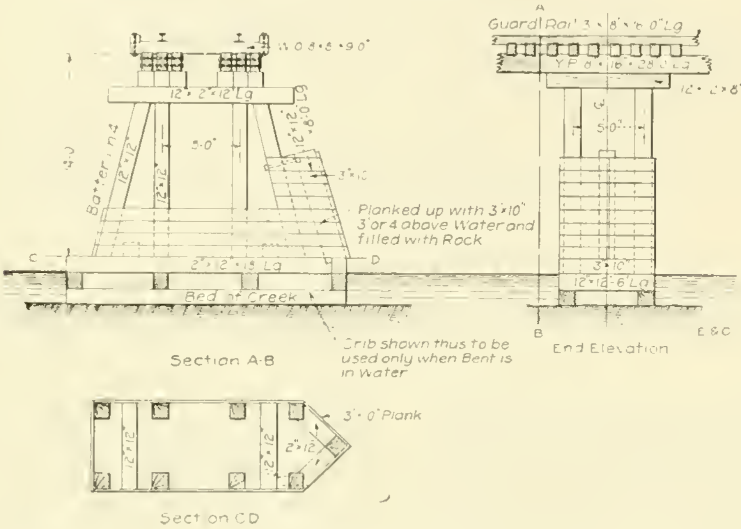


Fig. 10. Details of Timber Pier for 28-Ft. Trestle Spans Over Elk Creek, W. & Y. V. R. R.

Cost of Excavating a Rock Sidehill Cut.—Several large blasts have been successfully made along the route in sidehill cuts on different sections of the line, moving almost ½ cu. yd. of material for every pound of powder used. In one cut, estimated at 8,000 cu. yds., 95 per cent of which was solid rock, 33 holes were driven in two rows, the upper row being approximately 20 ft. deep and extending 2 ft. below grade, while the lower holes were 16 ft. deep, extending 6 ft. below grade. The holes were expanded or "sprung" twice, first by using 5 or 6 sticks of dynamite and then by using 25 or 30 sticks. This material was all rock and the second expansion of the holes was necessary, although it is believed that one expansion of the holes, using 10 or 12 sticks of dynamite, would have given better results, as the second expansion tended to fill the holes rather than open them up to sufficient size. An average of 11 kegs of power was used in expanding the upper holes and 13 in the lower holes. The results of this blast were very satisfactory, 378 kegs of powder being used, moving fully 7,000 cu. yds. of rock. A No. 3 push down battery was used in this explosion, although this was overloading the battery slightly.

Fig. 3 shows the profile of the cut where this blast was made and a plan showing the location, spacing and arrangement of the 33 blast holes.

Figure 4 shows a typical cross-section of the cut before and after the blast and as completed. A wagon road was shifted farther back from the river after the grading was done.

Figure 5 shows a view of the site of a small sidehill rock cut at Sta. 926. Holes are being drilled by hand for a blast that was made June 16, 1913. Six lower holes were drilled approximately 4 ft. down hill from the center line having depths of from 7 to 11 ft. Three holes were drilled along the center line, and 17 holes having depths of from 11 to 18 ft. were drilled at an average distance of 7 ft. above the center line. All holes were drilled about 2 ft. below sub-grade. The rock was of hard grey character, and eight cans of powder were placed in most of the deepest holes, although one hole required eleven cans. Three cans were loaded into the shallow holes. The cost of making the blast and cleaning out the cut, which contained a total of 1,300 cu. yds., was as follows, the item of $126 being for labor in cleaning out the loose material left by the blast and in dressing up the cut:

		Per cu. yd.
121 cans power at $1.30	$157.30	$0.121
150 lbs. dynamite at $0.15	22.50	0.017
50 20-ft. fuses	3.40	0.003
58 16-ft. fuses	2.78	0.002
Labor drilling and loading holes	135.00	0.104
Labor, 84 men at $1.50	126.00	0.096
Total	$446.98	$0.343

About 320 ft. of holes were drilled at a cost of approximately 40 cts. per lineal foot for drilling labor, which was only 0.24 ft. of hole per cubic yard of rock blasted. Powder was used at the rate of 2.3 lbs. per cubic yard of rock blasted, the springing of the holes requiring 0.12 lbs. of dynamite per cubic yard of rock. It was estimated that 900 cu. yds. of rock were shot out of the cut, so the unit cost for clearing out and dressing the cut was $126 ÷ 400 = $0.31 per cubic yard of rock left in the cut after the blast. Figure 6 shows the cut ready for track three days after the blast was fired.

Cost of Excavating a Rock Through Cut.—For the purpose of comparison the cost of excavating a through rock cut between Sta. 1038 + 50 and Sta. 1041 is here given. The material in the cut was medium hard rock, much of it being mica schist, and amounted to 2,200 cu. yds. Thirty-seven holes were drilled and fired in eleven blasts. About 28 pop shots and small shots were required for breaking up rock and removing rock to a point below grade. Some of the material was removed by wheelbarrows to the side of the cut, but the greater portion was moved by carts into a nearby fill, the haul being about 250 ft. The mules were owned by the company but the cost was estimated at $1.50 per 10-hour day. The total cost of excavating the cut was as follows:

Item.		
Mules and carts	$ 47.00	$0.021
Labor	1,018.25	0.463
Explosives	174.45	0.079
Totals	$1,239.70	$0.563

TUNNELS.

One short tunnel was constructed having a total length of 194 ft. As the rock was somewhat varying, in spots very hard and at other points loose, it was necessary to line the entire length.

The section adopted as shown by Fig. 7 is rather narrow, had it been the intention to maintain a permanent tunnel, but as it is the purpose to make an open cut at this point, the smallest safe section was adopted. The track has a summit about half way through the tunnel which provides for draining track ditches to both ends.

Cost.—The tunnel was driven by hand, the entire cost being $6,730.74, or $34.69 per foot. This is very reasonable, taking into account the fact that very little experienced labor could be found. No accidents in the tunnel proper occurred. The above cost includes all material, explosives and labor in any way connected with the tunnel. It required 39 days to drive the headings, and about the same time to drive the benches. Labor was scarce and difficult to get.

Method of Using Powder Tunnels for Excavating Rock.—The tunnel approaches contained approximately 40,000 cu. yds. of earth and rock, 75 per cent of which was earth. Nearly 50 per cent of this was moved by two blasts, one on each approach; 30,000 lbs. of powder were used. The powder in these later instances was placed in powder tunnels running about on the ditch line with branch tunnels leading from the main. The powder was emptied into paper flour sacks and then compacted as closely as possible in the extreme end of the main tunnel and branches. After the powder was placed, the balance of the tunnel was filled with earth compacted as well as possible. The fusees and lead lines in these blasts were tested at every step in the wiring of the blasts. Figure 8 shows the location of the tunnel on a curve and the plan of the powder tunnels. Figure 9 is a profile showing the amount of material moved by the two blasts in powder tunnels.

While effective and satisfactory work was done by these two blasts yet it is believed that by the use of more powder somewhat differently placed, had the dirt been better compacted at the tunnel mouth or had there been a solid stone or concrete wall built across the mouth of the tunnel, or in the tunnel next to the powder, better results would have been obtained. The powder tunnels were very dry and it is not believed the powder would have deteriorated very much during 48 hours necessary to give a concrete wall time to set. The fact that there was evidence of considerable force wasted at the mouth of the tunnel was evidence that there was quite a waste of good powder.

Total Amount and Average Cost of Excavation by R. R. Company Forces.—Approximately 24 miles were graded by company forces. The total yardage moved was 475,052, of which 99,688 yards were rock. Labor cost including explosives was approximately .12 ct. per yard for earth and 36 cts. for rock; 213,250 lbs. of powder were used and 24,000 lbs. of dynamite.

It is estimated that the powder moved from the grade, so that no further handling was necessary, at least 80,000 cu. yds. An additional large yardage of material was shaken up to be loaded by wheelers or loaded into carts or cars.

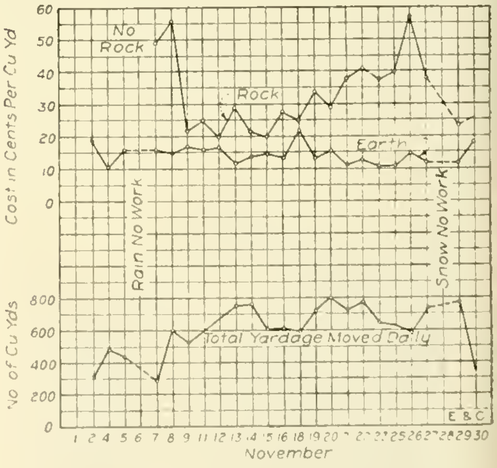


Fig. 11. Chart Showing Daily Movement of Excavation, Daily Costs of Earth and Rock Excavation, and Weather Conditions at Camp No. 2 for Month of November, 1912.

BRIDGES.

Pile and trestle bridges are too frequently constructed in order to save time, or to avoid real or supposed difficulty in forming embankments. The maintenance cost of such bridges is many times in excess of an embankment of equal first cost, and no bridge of this character should be built, unless the cost of embankment exceeds the first cost of such a bridge, and the subsequent cost of filling the bridge by work train or otherwise. The larger streams are bridged with trestles.

At some stream crossings the standard panel length of 14 ft. center to center of bents has been doubled to 28 ft. and timber piers constructed as shown by Fig. 10.

Corrugated culvert pipe was used up to 48 ins. diameter. Where larger openings were required trestle openings were built with the expectation of placing concrete culverts at such places at an early date. This was done to open the line to traffic as rapidly as possible and to secure a reduction of cost of transporting bridge and culvert material by the use of the railroad instead of teams. The tops of bridge stringers are set 0.25 ft. above regular grade on the profile, and regular grade changed about 100 ft. to obtain the 0.25 ft. raise.

REPORTS.

The foreman of each gang furnished a daily report of work done. Each gang was furnished with a counter, and some one man who had time registered the number of wheelers, scrapers, carts and cars or wheelbarrows moved. Where blasts were made, the amount moved was determined by measurement. These reports were sent in daily and the amount of dirt or earth moved at each camp charted and the unit costs determined. One of these charts for Camp No. 2 for the month of November, 1912, is shown by Fig. 11.

TRACK.

The track is laid with 70-lb. A. S. C. E. section rail. The joints are spliced with 24-in. angle bars using four 11/16 in. square nut bolts and ¼-in. nut locks. On curves of over 2° the rails were curved with an Emerson rail bender before being placed on the ties. The rails were laid with broken joints. The allowance made at joints for expansion was as follows: 100° F.—0 in.; 80° F.—1/16 in.; 60° F.—⅛ in.; 40° F.—3/16 in.; 20° F.—¼ in. and 0° F.—5/16 in. Ordinarily iron shims were used to secure the proper amount of space for expansion, but where the track was on steep grades sawed wooden shims were used which were left in the track until it was spiked, bolted and thoroughly anchored. The gage of track was made as follows: On tangent and on curves up to 4 degrees, 4 ft 8½ ins.; on curves from 4 degrees to 8 degrees, 4 ft. 8¾ ins.; on curves from 8 to 12 degrees, 4 ft. 8⅞ ins., and on curves from 12 to 20 degrees 4 ft. 9 ins. The superelevation of the outer rail on all curves is at the rate of ½ in. per degree up to a total of 5 ins. No greater elevation than 5 ins. is allowed. On grades exceeding 1 per cent no greater elevation than 4 ins. is alowed on curves of 10 degrees or over. The maximum superelevation on all curves of 15 degrees or over is 3 ins.

On straight track 18 ties per 33 ft. rail and on curves 20 ties per 33 ft. rail are used. Tie plates are used on curves of 4 degrees or over. The ties were tamped thoroughly with a shovel for a distance of 15 ins. inside of each rail and under the whole portion of the tie outside of the rails. The track is substantially planked at all road crossings.

Cost of Track Laying With a Track Machine.

(Contributed.)

In the first place, it is well to mention the fact that the conditions affecting track laying are numerous and varying, and it would be practically impossible for anyone except an experienced contractor with well organized forces to attain the results shown by the data below. Forces which have been employed for years in as narrow or specialized a field as track laying, are bound to attain a high degree of efficiency, provided the management is the best, for a body of most able overseers is gradually collected, which assures wise, efficient and progressive supervision. If, in addition to the above, a concern builds up a reputation for paying the best wages and giving absolutely the best treatment possible, consistent with the work required, an asset of no small importance is added.

The track laying described herein was done by a force which showed the results of all the advantages mentioned above. Every device or appliance which would help along the work was furnished. Tools and machinery were maintained at top efficiency through frequent expert attention. Years of experience have developed a construction organization of enviable efficiency. It is not surprising, therefore, that the costs appear exceedingly low, as compared with track laying by railway company forces. It should be noted also, in this connection, that in general rail laying by railway forces is done entirely by hand. The costs given do not include unloading the material in the material yard, loading it into the distributing train, nor transporting it to the "front."

One of the most vital reasons why contractors turn out cheaper work than railways can, is the fact that they offer better wages and thereby get better laborers in the end, as the weeding out process is available when good wages are paid. This leaves, for railway company laborers, inexperienced men who can not make good with contractors. Also stipulations are usually introduced into contracts which make the owner liable for extras in case the contractor's work is held up or interfered with. Consequently great efforts are made to keep the contractors' force fully supplied with material and room to work in. Thus the unit costs are greatly decreased through better planning, resulting in better and more rapid execution of the work.

MAKE-UP OF TRACK LAYING MACHINE TRAIN.

When laying track, the train carrying the machine is made up as follows, beginning with the "pioneer car," which always remains at the "front," and is not changed out as are the other cars in the train. Immediately behind the "pioneer" are four cars of rails, then the locomotive, and behind that eight cars of ties; next comes a car of tie plates, when they are used, the "trailer," which is a car carrying spikes, bolts and base plates, a car of plank for crossings, a car of cattle guards, a tool car and the way car. This makes twenty cars, and all are flat cars except the two last mentioned.

The first car of rail behind the pioneer is "trimmed," that is, on it is loaded angle bars enough to lay the amount of steel carried on the train. The angle bars are carried forward over the pioneer car and delivered as needed to the "strap hangers" in front. The rails underneath the angle bars are the last ones laid from the train, in order that the angle bars may be cleared off by the time rails are needed.

A system of trams is used to carry the ties and rails to the front. The trams are made in sections, each 33 ft. long, the sides consisting of 2½ x 10 in. planks. Tie trams are 14 ins. wide, and rail trams are 12 ins. wide. The trams are held together by bolts on which are pipe separators to hold the sides the proper distance apart. Near the bottom of the trams are live rollers, which complete a trough-shaped way for ties or rails.

On the pioneer car is installed a 20-h. p. upright engine for driving the live rollers in the trams; this is done by means of a tumbling shaft and gear or cog wheels. Steam for the stationary engine is piped from the locomotive. The shaft is fitted with "patent couplings," that is, on one end of each section is a casting containing a square socket into which the end of the next rod fits. Each length of tram has a section of the shaft bolted to it and as the trams are hung the rods are fitted together, thus forming a continuous shaft The trams are "hung" on iron brackets or trusses which hook into the stake pockets on the cars The trusses are made with flange rollers on which the trams are placed, thus taking care of the slack of the train in starting and stopping The trams have a coupling device which holds them together, the ones on the pioneer being permanently fastened to the car.

The tie trams, 660 ft long, are operated on the right hand side of the train Those for the rail, 210 ft long, are on the left The movement of ties and rail is controlled by the "dinkey skinner" i. e., the stationary engineer, so as to deliver them in front of the train as needed. A tie chute 53 ft. long provided with dead rollers is attached at the front end of the tie tram on the pioneer and through this chute the ties are pushed by the ones coming forward over the live rollers. And as fast as they are delivered at the end of the chute they are taken by the "tie buckers" (laborers) and are placed across the grade ready for the rails.

A similar chute attached to the rail tram provides a way for delivering the rail in front of the pioneer. These chutes are supported at the outer end by cables attached to the rear end of the pioneer car and carried up over a high frame work or "gallows" on the front end. A boom, also attached to the front end of the pioneer car, extends far enough ahead to have the cable attached to it reach the middle of the rail when placing it in position in track. This cable is operated by hand with an ordinary crab. Instead of cranks, a small, light buggy wheel is used by the operator to wind up the cable, which lifts the rail and holds it while the "heeler" and his assistants place it in position on the track. (A newer device handles the cable with compressed air). The rails are placed in the trams by three men, and are handled in front by six more. One man on each car places the ties in the trams. The spikes, bolts and base plates are peddled from the trailer as the train proceeds.

(The rails are held to gage by bridle rods until the train passes over, all spiking being done in the rear. The train moves ahead one rail length at a time when laying square joints, and half a rail length when laying broken joints. The trams are taken down when cars are empty and replaced on the loaded cars when a new train arrives; from 100 to 125 men are required for a full crew.

Material for the track machine is loaded by railway company forces, and great care is taken to have the material loaded, not only in correct proportion, but in correct order and position on cars. A train, called the swing train, is then made up of sufficient material for a half day's work, and is transported to the front, or rather to the camp of the contractor, where it is placed in the most convenient place available for the track machine crew to pick up. The swing train crew then takes a train of empties and returns to the material yard. The track machine is served regularly by the same locomotive and train crew. As the track machine does not move ahead by its own power, a locomotive and train crew are required to remain with the machine constantly.

Briefly, the movement of the machine is as follows in laying square jointed track: ties are trammed and carried ahead constantly and layed on the grade; the machine moves ahead, and a rail is chuted out and heeled in by the rail gang, and the angle bars bolted on loosely with two bolts only; a second rail is placed and held to gage by bridle rods; the machine is then moved ahead a rail length by the locomotive, and the operation repeated. When laying broken jointed track, the machine is moved ahead a half rail length at a time, thus requiring twice as many moves.

Back of the machine the bridle rods are removed, and enough ties are spiked to hold the rails from spreading. Spacing ties, bolt tightening and full bolting are all done behind the machine, and cause it no delay.

ORGANIZATION OF GANG.

A gang of 125 men will easily lay two miles of track per day, provided no unusual difficulties, such as soft grade, etc., are encountered A gang of this size would be placed about as follows:

1 general foreman at, per day..........	$ 5.00
1 ass't foreman, with rail gang, at, per day	3.50
1 ass't foreman, watching trams, at per day	3.50
1 ass't foreman, with spikers, at, per day	3.50
1 ass't foreman, lining track, at, per day	3.50
1 stationary engineer at, per month......	75.00
1 pole man at, per month..............	75.00
1 oiler at, per day	2.50
1 line man at, per day................	2.25
16 "tie buckers" at, per day......$2.25 and	2.50
2 tie spacers, ahead of machine, at, per day	2.25
1 man fiddling ties at, per day...........	2.25

6 "rust eaters," handling rail, at, per day	2.50
1 bridle man at, per day	2.25
1 heel nipper at, per day	2.25
2 strap hangers at, per day	2.25
1 man, carrying angle bars from "trimmed" car to pioneer car, at, per day	2.25
3 steel rollers, rolling rails into trams, at, per day	2.50
8 tie trammers, rolling ties into trams, at, per day	2.25
2 spike peddlers, distributing spikes, at, per day	2.25
2 bolt and joint plate peddlers at, per day	2.25
2 "bridle men," carrying bridle rods from rear, at, per day	2.25
4 rear bolters at, per day	2.25
2 water boys at, per day	2.25
8 men spacing ties at, per day	2.25
1 gage man at, per day	2.25
32 spikers at, per day	2.50
16 nippers at, per day.....$2.25 and	2.50
8 liners at, per day	2.25

When the gang is smaller, the force behind the machine is cut down, and 70 men would be organized about as follows:

1 general foreman at, per day	$ 5.00
1 ass't foreman, with rail gang, at, per day	3.50
1 ass't foreman, watching trams, at, per day	3.50
1 ass't foreman, with rail gang, at, per day	3.50
1 ass't foreman on general work, at per day	3.50
1 stationary engineer at, per month	75.00
1 pole man at, per month	75.00
1 oiler at, per day	2.50
1 line man at, per day	2.25
10 "tie buckets" at, per day......$2.25 and	2.50
2 tie spacers at, per day	2.25
6 rail handlers at, per day	2.50
1 bridle man at, per day	2.25
1 heel nipper at, per day	2.25
2 strap hangers at, per day	2.25
1 man carrying angle bars at, per day	2.25
3 steel rollers at, per day	2.50
8 tie trammers at, per day	2.25
2 spike peddlers at, per day	2.25
2 bolt and joint plate peddlers at, per day	2.25
1 bridle rod man at, per day	2.25
2 rear bolters at, per day	2.25
1 water boy at, per day	2.25
1 gage man at, per day	2.25
4 men spacing ties at, per day	2.25
12 spikers at, per day	2.50
6 nippers at, per day	2.50

During the work from which the cost data were obtained, the gang varied from about 50 to 100 men. The $2.50 laborers (spikers, nippers, and tie buckers) averaged about 40 per cent of the entire gang. During the 65 days the following expenses were chargeable against track laying:

Overhead charge on machine (interest at 6 per cent, depreciation at 10 per cent)	$ 100.00
Dinkey skinner, 2½ mos., at $100	210.00
Timekeeper, 2½ mos., at $85	177.00
Locomotive and crew, 65 days, at $40	2,600.00
Supervision and labor	8,710.00
	$11,797.00
Force account, or extras allowed	578.00
	$11,219.00
Average cost per mile	$ 280.50

This cost represents the cost to the contractor, plus the cost of the locomotive and crew at $40 per day. The latter charge should be added, however, as it represents a real part of the operation expense of the track machine.

GENERAL TRACK CONDITIONS.

The organization of a force working on a track laying machine is easily adjustable to the amount of laborers at hand, within certain limits, by a foreman who is competent. When a full crew is not available, the man in charge will cut out certain parts of the work, such as full spiking and bolting behind the machine, reduce the number of the "tie buckers," viz., men carrying ties to the minimum, take off the lining gang from behind, and so on all through the entire crew wherever a man can possibly be spared, leaving only those laborers whose work is absolutely necessary to be done while the rail is being laid. The rest is left to be done on the "back work." The two methods given above illustrate this. Occasionally a crew becomes so small when men are scarce, that only half of a train will be laid in a half day. This is an expensive method for the contractor as it generally necessitates "taking down" and "hanging" the trams an extra time for a mile of track.

When a full crew is on the work a mile of track can be easily laid in from three to four hours, including hanging and taking down the trams.

The track from which the cost of track laying above was computed, was laid during the winter months, and some bad weather was encountered, but the work probably progressed as fast as it would in the summer months, when extremely hot weather is liable to slow up the men.

The rail used was the standard length 33 ft., laid square joints on tangents and broken joints on curves. When a curve was reached a rail was cut to break the joints, the cut being figured so that the short part was used on the inside of the curve at the start, and then the long part was used at the end on the outside of the rail of curve to square the joints for the tangent. The specifications stipulated that joints must not be laid within four feet of the ends of bridges and culverts. To avoid cutting rails to meet this condition, fractional steel (short rails) was loaded on the "trimmed" car and when approaching a bridge the distance would be measured, and if found necessary a panel or more of these short length rails would be used to bring the joints the desired distance from the end of the bridge. In laying through yards where sidings were located, the main line was laid through regardless of the switches, and when switches were put in they were laid as near to the engineer's location as they could be put without cutting a main line rail. The fractional steel, a certain amount of which all companies agree to take with every large order for rail, was laid between the switches on the main lines through station grounds. As a rule the sidings were all laid with relieved rail, the work being done by hand.

The rail was a 90-lb. section, that is, it weighed 90 lbs. per yard. It was laid on white oak ties, spaced 18 to 21 under a 33 ft. rail on tangent, and 19 to 22 ties per 33 ft. on curves. The joints were made with ordinary angle bars with four bolts, and spring nut locks. The heads of the bolts were staggered, that is, alternate bolt heads were respectively on the inside and outside of rail. The number of ties per rail length were varied to suit their sizes—18 broad faced ties being used, or 21 narrow faced ties, on tangents.

The cost of transporting the machine and the men to the work is not included herein, the data given representing the costs after the machinery and the laborers were on the work.

An inspector was employed by the company, but although his expenses represent a charge against the track by the railway, it is not chargeable against the contractor's expenses.

COMPARISON.

The scarcity of costs on track laying make definite comparisons rather meager. In 1905 and 1906, however, the cost of laying track on the Erie R. R. was shown in an article by H. C. Landon, in the Erie R. R. Employes magazine. The cost of laying track was said to be about $200 per mile. The cost of labor and supervision was as follows, for a gang of 60 men:

1 foreman at, per day	$ 4.00
2 ass't foremen at, per day	3.00
Locomotive and crew at, per day	40.00
61 men at, per day	1.50
1 water boy at, per day	1.25

The average cost of labor in the 1913 work described herein was 58 per cent above that of the 1905 work; while the cost of tracklaying in the 1913 work was only 40 per cent above that in 1905. It is probable that the steel laid in 1905 was a much lighter section, and possibly the ties were soft ties spaced at wider intervals. It is safe to say that with the present class of average railway company labor, it would be impossible to lay track for less than $350 per mile, and $400 is probably paid in a good many cases.

Drainage and Irrigation

Prevailing Types of Pumps in California and Their Suitability for Irrigation Work.

Contributed by C. R. Sessions, San Francisco, Cal.

A splendid market for pumps driven by electric motors and gas engines has been opened as the result of the great development of hydro-electric power, an abundance of cheap distillates of crude oil, and the profit that comes to the farmer who irrigates California land. Conditions prevailing in different parts of the state determine the type and size of pumping plant that is used and that may range from a popping, air cooled one-horsepower gas engine driving a small pump jack to supply water to a few stock, to some one large drainage plant, having a capacity of as much as 350,000 gals. per minute, lifting the seepage and runoff from the hills up over a levee and into a river that during floodtime is booming along and carrying an enormous amount of water high above these rich delta lands.

The piston type pump is most efficient for higher heads and small quantities of water and for this reason the great first cost of displacement pumps does not prevent their use by the small gardener or the orchardist whose water supply is a deep well. When, however, something more than 150 to 200 gals. per minute is required, a centrifugal pump is generally used because it is relatively cheap, easily installed, and has an efficiency approaching that of the displacement pump as the quantity is increased. Irrigation is expedited by a large head of water and unless the irrigator is forced into using less his aim is to work as large a head as he can control. Excessive minimum rates charged by the power companies on large motors and the large first cost of a big plant do not deter him from using a large head if he can get it. His mind is intent upon the success of his crop. The big plant gives him a chance to catch up if he is behind for any reason and gives him a comfortable feeling of security so that he can handle his crop to advantage. Competition may force upon the irrigator the use of the smaller plant working more nearly continuously, but in California for the present arguments for refinement offered by power companies who want to keep their load factor as high as possible are met with the statement that the profit is in the crop and that the operation of the pump is a secondary matter and that the large plant is preferred.

The centrifugal pumps used in California for irrigation are nearly all supplied by local manufacturers. Eastern pumps have not been able to find their way into this market in the larger sizes because they can not be stocked very well upon the coast and in the smaller sizes because they are not what the market wants. The local pumps have undergone a rapid development, so that the eastern pump by the time it is stocked on the coast is not just what the market demands. The opening of the Panama Canal and the standardizing of centrifugal pump design may some day open this market to the eastern pump manufacturer. At present the local man has the advantage and is holding it by selling special pumps and making quick deliveries. He can supply his pump with special discharge and suction fittings and furnish a runner of an odd diameter and capacity to suit special conditions under which a stock pump would not operate satisfactorily. This is particularly so in the case of the centrifugal pump directly connected to n electric motor. Here the runner must be carefully designed so that the motor will not be overloaded and at the same time have speed enough to throw the right quantity of water at the highest head. To design and furnish uch a runner the manufacturer must know xactly the conditions under which the pump s to operate and have a sufficient number of patterns to choose from, so that he can get out the pump promptly and at not too great in expense. From his experience and his knowledge of local conditions he can go beyond the information furnished by a customer concerning the head under which the pump is o operate and supply a pump that will give satisfaction, while the eastern man who has not this information must rely upon a contract o get him his money when it is found out that while the pump is correctly designed for the head as given by the customer, the head is not actually what the customer thought it would be and as a result the pump is not just right for some cause that it will cost money to remedy. The manufacturer can not be blamed, but unless he makes the pump right without cost to the customer the next pump will be bought from a local manufacturer who has the reputation of furnishing pumps that work just right. To illustrate this a case that often happens may be cited. A farmer wants a pump directly connected to a motor to pump from wells which he has had drilled but which he has not carefully tested. He is led to believe by the well borer and his desire for good wells that a certain quantity of water will be got with a certain pull on the wells. Now it may happen that this quantity and head just about fit a motor of a certain horse power and speed. The eastern manufacturer would furnish his runner to suit these conditions only while the local man, knowing that the wells in that vicinity do not generally show up as well as the farmer figures would make his runner a little larger, so that when the outfit is installed all the water that can be in reason expected will be discharged by the pump. If, as is rarely the case, the runner is so large as to overload the motor then at small cost, it can be reduced in diameter to suit without cost to the farmer. If the other pump were installed not enough water would be discharged and the farmer would be led to believe that the eastern pump was as the local manufacturer claimed, not equal to the local pump in the amount of water that could be discharged. Any reflection on the wells makes an enemy of the well borer, who is some talker and who may for financial reasons be a very good friend of the local pump manufacturer.

The features that distinguish the local pumps are whether or not the runner is open or enclosed, whether the axial thrust on the runner is balanced by a mechanical thrust bearing or by an automatic hydraulic device and flexibility of the runner as regards range of head for a given capacity. Mechanical features, such as ring oiling bearings, water sealed stuffing glands and split casings, distinguished some of the local pumps from the others. The open runner or impeller is to be found in the cheaper and older pumps. This runner depends upon the running fit between the runner and the side plates of the pump casing to prevent leakage from the discharge back into the suction. As the pump wears this fit is impaired, and as a result, the efficiency is lowered steadily as more and more water is by-passed. The enclosed impeller or runner, with sides extending from the hub to the rim, has largely superseded the open runner. Here the leakage is prevented by the fit between a recess in the casing and a shoulder on the hub of the runner. This shoulder is of relatively small diameter, so that the leakage area is not great. When the water contains sand and wear is apt to be excessive, removable bushings are fitted that can be easily replaced and at relatively small expense. The fact that the suction opening on a single suction pump causes a thrust in the direction of the suction opening, varying in amount in proportion to the area of the opening and the vacuum or suction head, makes it necessary to have some sort of balancing device. In some devices this is accomplished by having a chamber on the opposite side of the runner that is connected by drilled or cored holes with the suction side, and that is separated from the pressure side or discharge of the pump by a hub running in a recess of a diameter calculated to take care of the effect of the area of the shaft and centrifugal action in the clearance spaces. This acts as a counter balance to remove the thrust for some particular head, but as the head changes the thrust changes, so that a thrust collar must be arranged outside the casing, and if the thrust is excessive, as it may be for a high vacuum, some means must be provided for cooling. It may be said here that when the impeller is thrusting steadily in one direction there can be no end play to the shaft, and as a result the stuffing box packing wears badly and must be compressed very hard to keep this gland tight. In the automatically acting hydraulic device the same two hubs are made use of, but the end of the hub is so fitted with reference to the recess that a slight lateral movement opens or closes the passage for water that will by-pass from the pressure to the suction side and causes an inequality automatically in the thrust to move the runner until the pressures on both sides are again equal. In this device the slight end play keeps the stuffing box packing in fine condition, and if some sort of an oil or water seal is used, little or no pressure is needed to prevent the air from working into the pump and causing it to lose its priming. The action of this balance is independent of the head against which the pump is working and about the only disadvantage argued against it is that the by-passed water lowers the efficiency of the pump, particularly if sandy or gritty water is being pumped.

It is not generally advisable to locate the pump farther than about 20 ft. from the water level, and this presents no great difficulty when the water is being pumped out of a river or lake or wells where the water level is not subject to much change, but when light wells are being pumped from it may be necessary to draw on the wells with a vacuum corresponding to lowering the water in the wells as much as 20 ft., which means that the pump must be placed as near as possible to the level of the water in the wells and before the wells are drawn upon, so that as large a suction head can be used as is possible. In other words, the pump must be put in a pit as deep as the surface water will allow. The pump, if it is a horizontal one, must not be under water. A vertical pump, however, can be put low enough so that the water may be drawn down 20 ft. farther than the distance that the dump is submerged. Such a pump makes it possible to draw very hard on weak wells and upon wells that may have their water level lowered toward the end of the season or by the fact that other irrigators are pumping from the same water stratum. The horizontal pump in such a case would have to be lowered as the water receded and would have to be raised again to keep it out of the water when the level came up in the winter or at the end of the season. This would be impossible if several wells were connected up by any great amount of piping. A water tight pit and a sealed well, although such a construction is possible, call for a grade of work that is seldom found in irrigating plants. The farmer can not see the necessity of building two walls of concrete just to make the inner one tight with asphaltum. He is sure to listen to someone who can put something in the concrete that will make it absolutely water tight. That something costs very little as compared with the cost of the pit, and he has paid for it long before he finds out that the pit is not tight and dry. A new type of vertical pump is being put on the market and may be said to have about passed out of the experimental stage. It is a deep well, vertical turbine pump of small outside diameter, which makes it possible to install it directly in the well casing without the use of a pit, the driven pulley being at the surface of the ground if it is to be belt driven and the motor being directly connected to the shaft at the surface of the ground if it is to be direct connected unit. These pumps have several stages, made necessary by the small runner diameters (one stage being good for about 20 ft.) and have the shaft made up in sections and carried in bearing inside a casing which forms the frame of the entire pump, and through which the water is discharged to the surface. Lubrication and alignment of this shaft is a problem, and the life of these pumps under hard usage is yet to be determined. They are expensive, but for the man who has a well that fails him in the latter end of the season they are a splendid means of drawing the last drop of water that can be got out of his well. This last drop is sometimes very valuable when the moisture is drying up fast and the fruit lacks but a short time of filling out and ripening, as it must be to be sold at a good price.

Lining Irrigation Ditches in Hawaii By the Cement Gun Process.

Some ditch lining data of interest to irrigation engineers are contained in an article by A. W. Collins, engineer, Maui Agricultural Co., in a committee report to the Hawaiian Sugar Planters' Association. This article, slightly condensed, is as follows:

The increasing value of water in various sections of the country in the last few years has led to many experiments in ditch linings, by both government and private capital. The prevailing idea has been to obtain a lining which is permanent and at the same time cheap enough to enable people with limited capital to do work otherwise prohibitive. Without doubt a 4-in. to 6-in. concrete lining is the best, but the cost is high. Rubble wall set in a lime mortar and then plastered with cement mortar has been tried with little reduction in cost, even getting lime at 75 cts. per barrel. Oil has been used in some sections, mostly in California, but this stops only a part of the seepage and would not be practicable for this country. There have been many plastered ditches, but results have shown them in the main temporary. Water gets in behind and soon causes the liming to crack off. This has been well illustrated on various ditches in the islands.

As far as I know Mr. Jorgensen on the Hamakua Ditch in Hawaii was the first to experiment with a plaster lining reinforced with chicken wire. A large part of the ditch was lined this way, and at the present time the work is standing up so well that it seems safe to call it an ideal lining. One big item is the elimination of all form work, which makes progress very slow and is expensive. The chicken wire is fastened onto the sides and bottom with nails or staples made out of an ordinary fence wire. In some places in rocky sections this is a little difficult but there are always enough cracks in the rock to be able to fasten the wire on in a few places. There are also places so uneven, as in shale or loose rock, that they require mudding before the wiring and plastering. The wiring and plastering should follow this mudding before it has had time to dry out and crack

The one difficulty is to get the plastering done well. With our ordinary island labor constant supervision is necessary. It is very easy to shirk the work, where the men are at all anxious to make a showing, by insufficient troweling. The mortar has a tendency to fall when first applied, and unless the worker is careful and experienced this causes a porous spot in the plaster. It is very difficult to make them rub it in thoroughly.

We became interested in the cement gun through its use by Captain Edwards in building construction at the different army posts near Honolulu. It was thought to be ideal for ditch lining to overcome the inefficiency of hand work. Both the Maui Agricultural Co. and the H. C. & S. Co. have a machine at present writing, but thus far only the one belonging to the former has been used.

There is a marked difference between cement mortar applied by hand and the product of the machine. The latter is shot under a pressure of approximately 10 lbs. to the square inch and leaves the nozzle at a velocity of 200 ft. per second, consequently it is far denser and of a much greater tensile strength that that applied by hand. One very important point in favor of machine applied mortar is that the combining of cement and water takes place in transit and the mixture does not have to be moved after an initial set. This is one of our worst troubles on concrete work in the islands. The laborers generally employed here do not realize the necessity of handling the mortar quickly after the water has been applied. I have seen them mixing so much at one time that it would be impossible to get it on inside of 30 to 40 minutes. This initial set, especially with some cement on the market, is considerable and is of course lost under these conditions. The men repeatedly do this thing unless watched, although explicit instructions to the contrary have been issued. This fault is entirely eliminated with the gun method.

Comparative tests of mortar made by hand and by the cement gun have been made by a well known engineering company. These show that the product of the cement gun developed from 20 per cent to 260 per cent greater tensile strength and from 20 to 720 per cent greater compressive strength than the product of the same mixture when applied by hand. These tests also showed that compared with the most carefully prepared hand made cement mixture, "gunite" is far more impervious and waterproof.

We have at this time finished 2½ miles of open ditch and three short tunnels, their respective lengths being 250, 400 and 740 ft. The outfit has been entirely satisfactory and very good results have been obtained. On our larger ditch we have averaged about 100 ft. per day, a running foot being equivalent to 18 sq. ft. All the labor on the job has been under contract, varying on this larger ditch from 15 to 20 cts. per linear foot. This cost covers mudding where the section is uneven, wiring, man for nozzle, men mixing sand and cement and supplying the gun, a helper for the engine and all labor for moving the outfit.

The tunnel work has given us a good line on what is possible with the gun. In all three tunnels the engine and gun have been at least 100 ft. from the end of the tunnel. This in the long tunnel would make an 850 ft. delivery pipe or hose from the gun to the nozzle. Working at this distance, however, diminishes the output considerably, since the air motor cannot be run wide open as when working nearer. This means less material delivered to the discharge pipe and consequently less work.

We are constructing two new ditches, Haiku and Kauikoa, 10 and 4½ miles long. These are both practically all tunnels, ranging from 800 to 3,000 ft. We expect to cement gun all possible. In some places where we have had cave-ins during construction or where the material looks dangerous we will concrete both sides and roof. There will also be places where there is so much water coming in that a mortar will be insufficient and concrete will be used.

We expect to work both compressor and gun outside of tunnel for the first 500 ft., delivering material in pipe or hose to nozzle. Beyond this distance the compressor will be kept outside and the gun will be moved into the tunnel. This will shorten the material hose and thereby lessen friction, which means we can run the air motor faster and accordingly can accomplish more work. The distance from compressor to gun—"the air line"—makes little difference as long as all joints are tight.

The gas engine driven compressors furnished with the outfit are excellent and would come in very handy for any riveting or light drill work. Everything is portable and little time is lost in moving from on position to another

The following is the record of complete cost of work for a period of five weeks on a ditch of 18 ft. cross section:

Labor, 2,600 ft. at 15 cts.	$ 390.00
Cement, 370 barrels at $3.50	1,295.00
Sand	100.00
Gasoline, 870 gallons at 22 cts.	191.40
Wire, 5,200 ft. 6-ft. at 4 cts.	208.00
Wire, 2,600 ft. 4-ft. at 3 cts.	78.00
Superintendence	150.00
Incidentals	100.00
Cost of 2,600 ft.	$2,512.40

Soil Porosity and the Distance Between Drainage Channels.

Contributed by Louis Schmeer, Long Beach, Cal.

In an article entitled "Groundwater-Movements," etc., published on Sept. 24, 1913, in Engineering and Contracting, the contributor gives values of K, the transmission constant, as derived from soil composition data published by the U. S. Geological Survey. A correspondent points out the fact that these values are erroneous as far as the surface layers of the soils are concerned, since the influence of the organic contents of the upper crust on the proposity is not taken into consideration. The position of the correspondent is well taken. Surface soils not entirely barren contain remnants of plant or animal life in greater or lesser abundance, the percentage in most cases rapidly decreasing with the depth. The products of the decomposition of plant life are usually present in the form of humus, a sponge-like substance of great but undetermined transmission capacity. Surface soils are moreover exposed to the physical activities of the atmosphere, to heat, cold, drying winds, etc., which tend to increase their porosity in a great degree, a fact which is most plainly evident in the case of the so-called joint clays rendered porous by repeated wettings and dryings. When a soil is subjected to a mechanical analysis, its natural porosity is destroyed and its humus contents are eliminated; a true estimate of its transmission capacity is in consequence impossible. Reliable hydrological values can only be found by actual experiments in the field and none thus derived are at present available.

A simple device for the determination of the transmission capacity of the soil *in situ*, denoted lysimeter, is made and operated as follows: A steel pipe 2 ft or over in diameter and having a sharp cutting edge is driven into the soil to the desired depth; the soil enclosing the pipe is then removed and the interior soil column completely isolated by driving a sharp-edged steel plate across the bottom of the pipe. Water is then admitted to the surface and the supply so regulated that a thin sheet always covers the soil column. As soon as the discharge at the bottom is regular it is measured and the value of the transmission constant found from the equation.

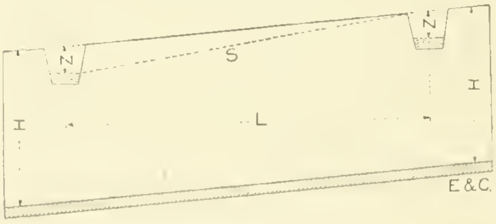


Fig. 1. Cross Drainage.

$$K = \frac{\text{Discharge in cubic feet per hour} \times 24}{\text{Area of cross-section in square feet.}}$$

In part II, Report of U. S. Geological Survey of 1898 Prof. King gives data describing the hydrological properties of some Wisconsin soils from which fairly reliable values of the transmission constant may be computed. The effective diameter of the soil grains was determined by means of an apparatus known as King's aspirator. This device permits a determination without the elimination of organic substances and the values obtained are in consequence more accurate than those derived by other methods.

In Table I, columns two and three give the porosity percentages and effective diameters as determined by King and column four gives the corresponding values of K computed by the writer from the formula

$$K = 22464\, P^{3.3} D^2$$

It will be observed that in the sandy soils the surface layer is the least permeable while in the heavy soils the reverse is the case. This phenomenon is due to the fact that the weathering process converts the coarse sand grains near the surface into the less permeable silts while those at greater depths are not affected. In the clays, representing the final product of the decomposition of many minerals, the soil grains are more nearly alike throughout the mass and the presence of organic substances in the surface layers renders these the most permeable.

Summarizing the values given in column 4 we find:

For the heavy clays, 2 samples—	$K = 0.20 - 0.30$
For the clayey loams and loamey clays, 4 samples	$K = 0.60 - 0.80$
For the lighter loams, 2 samples.	$K = 0.90 - 1.50$
For the sandy soils, 10 samples.	$K = 0.50 - 7.75$

It is probable that somewhat higher values would be obtained for the same soils found *in situ*. Having the true value of K, the distance between drainage channels is easily found.

Let $D.P.$ = the average maximum daily precipitation in inches.

H = the elevation of the surface of the soil (or of the watertable) above an impervious layer.

Z = the depth of the drainage channel (or, more accurate, the elevation of the soil surface above the water in the channel)

then from the formula

$$Q = \frac{KsHZ}{2}$$

putting $\frac{Z}{2L} = s =$ *the mean slope,*

$\frac{Z}{2}$ = the mean depth of the layer through which the water penetrates into the channel.

$$D.P. = \frac{KH}{2}\,\frac{Z}{2L}\,\frac{Z}{2}\,\frac{12}{L}$$

which reduces to

$$D.P. = \frac{1.5\,K\,H\,Z^2}{L^2}$$

Hence

$$L = Z\sqrt{\frac{1.5\,KH}{D.P.}}$$

TABLE I.

Description of soil—	Porosity. P	Effective diameter. D	K
Sandy soils:	0.3449	0.02619	0.49
Minong, Wis., surface	......	....	....
Minong, Wis., 6-12 ins.	0.2996	0.0523	1.15
Cypress Point, Wis., surface	0.3230	0.03149	0.54
Cypress Point, Wis., 8-16 ins.	0.3230	0.09352	4.72
Tomahawk, Wis., surface.	0.3457	0.04751	1.51
Tomahawk, Wis., 12-18 ins.	0.3143	0.1242	7.72
Superior Junction, Wis., surface	0.3619	0.03439	0.93
Superior Junction, Wis., 6-12 ins.	0.2932	0.3589	0.48
NeKoosa, Wis., first foot.	0.4068	0.02501	0.72
NeKoosa, Wis., second foot	0.3287	0.05197	1.54
Prairie loam, Oasis, Wis., surface	0.3883	0.03035	0.91
Prairie loam, Oasis, Wis., second foot	0.3464	0.04777	1.56
Clayey loam, Drummond, Wis., surface	0.4487	0.02206	0.78
Clayey loam, Drummond, Wis., second foot	0.4415	0.02197	0.73
Clayey loam, Bruce, Wis., second foot	0.4827	0.01804	0.66
Loamey clay, St. Croix, Wis., second foot	0.4710	0.01810	0.61
Heavy clay, Ashland, Wis., 1-6 ins.	0.4532	0.01402	0.32
Heavy clay, Ashland, Wis., 6-8 ins.	0.4415	0.01111	0.19

In cross drainage, where channels run at right angles to the greatest slope a small quantity of water will percolate into the upstream channel. The dividing line between the movement towards the lower and that towards the upper channel is evidently given by the line of equal velocities. Let

X = the distance discharging into the lower channel,

Y = the distance discharging into the upper channel, then, for equal velocities

$$\frac{K}{P}\,\frac{H}{2}\,\frac{Z}{X} = \frac{K}{P}\,\frac{Z}{2}\,\frac{Z}{y}$$

which reduces to

$$X = \frac{H}{H+Z}\,L$$

Hence for cross discharge

$$D.\,P. = \frac{1.5\,KH\left(\frac{H+Z}{H}\right)^2 Z^2}{L^2}$$

In agricultural soils, excluding the sands, which rarely need drainage the subsoil is the denser and more compacted the greater the distance from the surface and in the lower zones it is practically impervious. The depth at which porosity vanishes may be assumed to be about 16 ft.

Drainage experts in Europe and the United States are practically agreed that the basis of calculation in drainage work should be an average maximum daily precipitation of 6 millimeters or 0.24 in.

Assuming $H = 16$ and $D.\,P. = 0.24$ and inserting values

$$L = Z\sqrt{\frac{1.5\,K16}{0.24}}\left(\frac{H+Z}{H}\right)$$

which reduces to

$$L = 10\,Z\sqrt{K}\left(\frac{H+Z}{H}\right)$$

This holds good when channels are run at high angles to the greatest slope; in other words, where cross-drainage is in use.

For longitudinal drainage where channels run in the direction of the greatest slope, a strip between two laterals is drained by both channels.

Hence

$$s = \frac{2Z}{L} \text{ and } H = Z.$$

consequently

$$L = 5\,Z\sqrt{KZ}$$

For $Z = 4$ ft., the usual depth for tile drains, it is seen that for cross-drainage $L = 50\sqrt{K}$, for longitudinal drainage $L = 40\sqrt{K}$.

If the values of K, based on King's experiments, given above, are inserted in these equations, the resulting values of L are substantially those long ago found suitable in practice.

In open channel drainage the depths of the cuts made by ditching machines or dredges frequently far exceed the depth to which it is necessary to deprive the soil of its surplus moisture.

Let d be the depth to which the soil is to be drained then, the mean slope, neglecting the slope of the land

$$s = \frac{Z - \frac{d}{2}}{L},$$

the relative depth to be drained $= \frac{d}{Z}$, which gives for cross-drainage

$$L = 3.53\left(\frac{H+Z}{H}\right)\sqrt{KH\left(\frac{Z^3}{d} - \frac{Z^4}{2}\right)}$$

for longitudinal drainage

$$L = 7.07\sqrt{K\left(\frac{Z^4}{d} - \frac{Z^3}{2}\right)}$$

Since in the first of these equations the influence of the slope of the land has been neglected, the true value of L must be found by a second trial giving in this to Z an increase equal to the rise of the ground in the distance L.

The depth of the channel for a given distance L is found by putting, for cross drainage

$$\left(Z^3 - \frac{d}{2}Z^2\right) - \frac{d}{KH}\,\frac{L^2}{12.5}\left(\frac{H}{H+Z}\right)^2 = 0.0$$

for longitudinal drainage

$$\left(Z^4 - \frac{d}{2}Z^3\right) - \frac{d}{K}\,\frac{L^2}{50} = 0.0$$

which may be solved by Horner's method or by successive trials, putting in the first case

$$Z = \sqrt[3]{\frac{d}{KH}\,\frac{L^2}{12.5}\left(\frac{H}{H+Z}\right)^2 + \frac{d}{2}Z^2}$$

in the second

$$Z = \sqrt[4]{\frac{d}{K}\,\frac{L^2}{50} + \frac{dZ^3}{2}}$$

giving to Z in the terms on the right hand side for a first trial an assumed value. Suppose a large tract of comparatively level land is to be drained by means of open cuts, which, in order to waste as little land as possible, are to be not less than one-sixth of a mile or 880 ft. apart. The soil, a sandy loam, is to be drained to a depth of 3 ft. and the value of K is found or assumed to be equal to 6.0. Inserting these values in the last equation, assuming for a first trial $Z = 10$ ft.

$$Z = \sqrt{\frac{3}{6}\,\frac{880^2}{50} + \frac{3\,.\,10^3}{2}}$$

or, $Z = 9.8$ ft. Hence no second trial is required. The first equation gives $Z = 9.02$ ft. for $H = 16$ ft., but from this the rise of the ground in the distance of 880 ft. is to be subtracted.

This value of Z, given by the first equation, indicates, however, that for the deeper cuts, those exceeding 6 to 8 ft., cross-drainage offers special advantages only in case the slope of the ground is considerable.

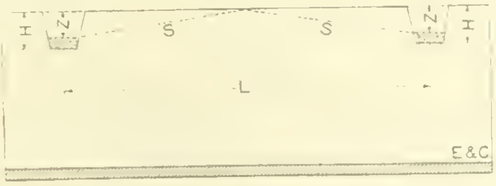


Fig. 2. Longitudinal Drainage.

In all of the preceding equations the precipitation has been assumed to be equal to 0.24 in. per day, corresponding to double a maximum monthly precipitation of 3.6 in. In localities where this is much exceeded distances between drains as given by the formulas are to be multiplied by

$$\sqrt{\frac{3.6}{M.\,M.\,P.}}$$

M. M. P. being the maximum monthly precipitation recorded during the month of the greatest precipitation, usually a month in the early spring.

The table below gives values of

$$7.07\sqrt{\frac{Z^4}{d} - \frac{Z^3}{2}}$$

corresponding to the different depths Z of the cuts and the different depths d to which the land is to be drained. For the actual distances between drains these values are to be multiplied by $\sqrt{K}$ for longitudinal, and by $0.5\,\frac{H+Z}{\sqrt{H\,.\,Z}}\sqrt{K}$ for cross drainage.

For the depths to which it is desirable to deprive the soil of its surplus moisture the following distances have been found suitable in practice:

	Ft.
In meadows and pastures	2 —2.5
In agricultural lands	3 —4
In garden lands	3.5—4.5
In vineyards and orchards	4 —5

In the absence of complete experimental data the contributor suggests the following values of K and $\sqrt{K}$:

Description of soil.	Percentage of soluble matter.	K.	$\sqrt{K}$.
Stiff clay	75—90	0.25	0.5
Common clay	50—75	0.75	0.86
Heavy loam	40—50	1.15	1.07
Medium loam	30—40	1.5	1.22
Sandy loam	20—30	2.0	1.41
Loamy sand	10—20	3.25	1.80
Silty sand	0—10	5.00	2.23
Barren sands	......	36.0	6.0

Cost of Reservoirs for Flood Control.

To the Editors: In the writer's paper on flood prevention read before the Indiana Sanitary and Water Supply Association and published in your issue of March 18, 1914, there is a paragraph discussing the costs of reservoirs illustrated by the cost of the Pittsburgh system. I find that I made an error in quoting the drainage area of the rivers above Pittsburgh: I took the data from a congressional document, the statement coming from the Chief of Engineers' Office, so that I did not look up the matter further. I find that the area quoted is in error, and therefore the figures following are incorrect. Change to read as follows:

> Besides the points mentioned above, there enters the question of costs. Reservoir sites are, in general, extremely costly. The Pittsburgh Flood Commission estimated the cost of its proposed system at $21,000,000, but a Board of Engineers, U. S. Army, in reviewing the report found that land and property damages were underestimated by $13,000,000, so if their other estimates were correct, the cost would be $34,000,000. If a complete system were practicable for the Ohio River, some idea of its probable magnitude may be gained from the fact that the area of the basin of the Ohio Valley is 204,320 square miles, while the drainage area of the rivers above Pittsburgh just referred to is but 18,920 square miles, or 9 per cent of the whole. Assuming that the cost per square mile would be the same as that for the Pittsburgh system, the total cost of providing reservoirs for the Ohio River would be $378,000,000. It is probable, however, that these costs would be greater, for the territory in question above Pittsburgh is not nearly as valuable as in such states as Ohio, Indiana and Illinois.

Very truly,
J. C. Oakes,
Major Corps of Engineers.
Louisville, Ky., March 18, 1914.

Depth of channel. Z.	Values of d in feet 2.	2.5	3.	3.5	4.	4.5	5.
	Values of $7.07\sqrt{\frac{Z^4}{d} - \frac{Z^3}{2}}$						
2	14.1						
2.5	23.1	[illegible]					
3	36.8	[illegible]	[illegible]				
3.5	51.7	[illegible]	[illegible]	[illegible]			
4	69.3	[illegible]	[illegible]	[illegible]	[illegible]		
4.5	89	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	
5	111.0	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	55.8
5.5	[illegible]	117.8	[illegible]	[illegible]	[illegible]	[illegible]	70.7
6	161.0	[illegible]	[illegible]	[illegible]	[illegible]	94.8	87.0
6.5	191.3	169.7	[illegible]	[illegible]	[illegible]	[illegible]	104.8
7	[illegible]	198.6	173.2	[illegible]	[illegible]	[illegible]	124.2
7.5	[illegible]	[illegible]	[illegible]	[illegible]	170.2	[illegible]	145.2
8	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	180.8	167.8
8.5	[illegible]	298.4	[illegible]	[illegible]	[illegible]	206.4	191.5
9	[illegible]	[illegible]	291.8	241.7	[illegible]	[illegible]	217.6
9.5	[illegible]	[illegible]	[illegible]	308.0	[illegible]	[illegible]	244.9
10	474.2	[illegible]	[illegible]	[illegible]	316.7	[illegible]	273.8
10.5	[illegible]	[illegible]	416.6	[illegible]	350.7	[illegible]	304.1
11	[illegible]	509.1	[illegible]	[illegible]	[illegible]	[illegible]	336.2
11.5	[illegible]	[illegible]	[illegible]	460.1	421.9	395.2	369.8
12	[illegible]	[illegible]	[illegible]	[illegible]	464.7	[illegible]	405.1
13	[illegible]	[illegible]	[illegible]	594.1	549.3	511.9	480.0
14	945.2	[illegible]	[illegible]	[illegible]	641.2	[illegible]	561.6
15	1086.6	962.9	871.9	799.4	719.9	691.3	649.1
16	[illegible]	1098.6	991.7	[illegible]	816.3	790.1	743.9

Sewerage

Design of the East Side Sewage Pumping Station, Hartford, Conn.

Certain portions of Hartford, adjacent to the Connecticut River, have always been subject to flooding. Various attempts have been made during the last half century to prevent the inundation of these areas by the construction of dykes or by raising the low lying areas throughout what is known locally as the East Side. It was not until 1908, however, when the improvements in connection with the approaches to the river bridge were carried out that any material progress was made. At that time the construction of a boulevard and the raising of a street by the city, accompanied by the construction of a freight yard and embankments by a railroad completed a dyke for a portion of the area subject to floods.

The dyked area was still subject, however, to the backing up of water in the sewers which pierced the dyke at each street. In 1910 an appropriation of $200,000 was made by the city to build an intercepting sewer to collect all sewage and storm water from the dyked area and also, eventually, from the now undyked area, and a pumping station to dispose of this runoff at times of high water in the Connecticut River. The appropriation was increased by $50,000 in 1912. It is to the design of the pumping station that the present article chiefly relates, the article being based on information given in the paper by Mr. W. S. Brewer, assistant city engineer of Hartford, before the recent annual convention of the Connecticut Society of Civil Engineers.

The intercepting sewer was built in 1910-11 and all sewers in the district were connected to it, the old channels leading to the river being blocked off. The street previously mentioned as having been raised by the city (State St.) is the southern boundary of the dyked section. It was necessary to construct a sanitary sewer in that street to take the sewage from all buildings on the north side of the street of which the cellars were below flood level. The surface drainage and that from the buildings on the south side still goes into the old sewer which continues directly to the river, crossing the interceptor above it. No sewers or drains were connected to the new interceptor south of State St. and the manholes were provided with watertight covers. This action was necessary in order to prevent any drainage or leakage reaching this portion of the sewer when the land and streets above it are flooded. When a dyke is built to protect the remaining area now subject to flooding these sewers which now continue to the river will be connected to the interceptor.

DESIGN OF STATION.

Capacity.—The area subject to floods which will eventually be drained by pumping amounts to 160 acres. Using the formula $Q=cr\sqrt[5]{A^4s}$, with $c=70$ per cent and $r=2.7$, the runoff amounts to 217 cu. ft. per second. The lower end of the interceptor was designed of this capacity. It was considered that a severe storm would seldom occur at the time of maximum height of the river so it was thought safe to design the pumping station with less capacity, and the pumps as installed have a rated capacity at maximum head of about two-thirds this amount.

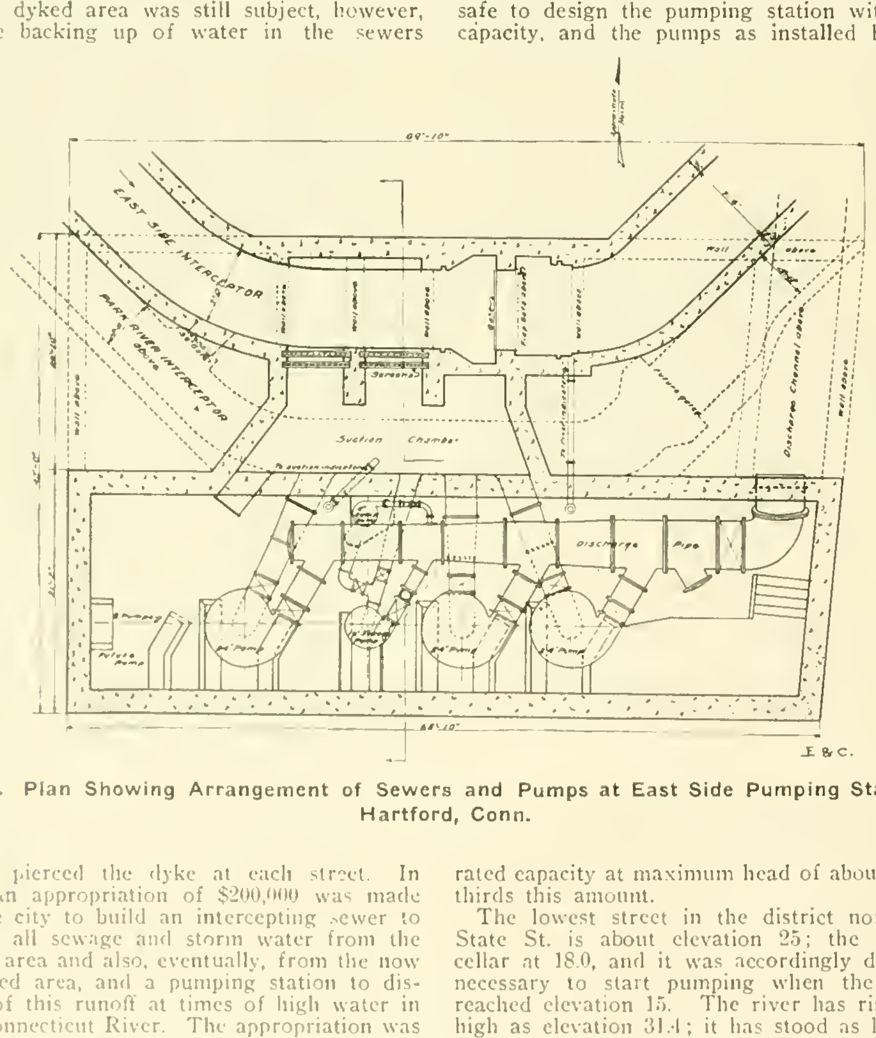


Fig. 1. Plan Showing Arrangement of Sewers and Pumps at East Side Pumping Station, Hartford, Conn.

The lowest street in the district north of State St. is about elevation 25; the lowest cellar at 18.0, and it was accordingly deemed necessary to start pumping when the river reached elevation 15. The river has risen as high as elevation 31.4; it has stood as low as elevation 1.0. Its mean elevation during the dry weather period may be considered as about 4.0. The following gives the average conditions for the period from 1896 to 1907:

River above elev. 15, average of 28 days per year.
River above elev. 20, average of 9 days per year.
River above elev. 25, average of 1 day per year.

It is thus seen that it will be necessary to pump the sewage flow for an average of 28 days each year and from the rainfall records during the last 11 years it would be necessary to pump storm water an average of only 5 days each year.

Another consideration which affected the location and design of the station was that at some future date, to reduce pollution in the river at the present sewer outlets it will be necessary to pump all sewage to filter beds. When this becomes necessary small automatic pumping stations will be installed to pump the sewage from certain present sewers into the interceptors which run to the main sewage pumping station here under consideration. By this method all sewage would be removed from the river front. In times of storm the flow would pass by the pumps direct to the river.

The general considerations which governed the location and design of this pumping station may then be stated as follows: It must dispose of all sewage and storm water brought to it by the East Side interceptor when the river rises to or above elevation 15.0, and it must dispose of all sewage brought to it by both the Park River and East Side interceptors when at some future time it is considered desirable to free the river frontage from sewage.

The station is a reinforced concrete and brick building 42 ft. by about 68 ft. in plan and 55 ft. in height from the lowest part to the top of the roof coping. The surface in front of the station is at elevation 24 and as the elevation of the bottom of the concrete of the sump pit is —2, and that of the coping 53, it will be seen that there is nearly as much of the station below ground as there is above. The main floor is at elevation 34, 2.6 ft. above the highest recorded flood. As all electrical apparatus and operating gears (except the gate on the discharge from the sump pump) are on this floor, the station could still be operated even though for any reason the flood waters of the river gained access to it and flooded it to the level of the water outside.

All construction below this floor is of reinforced concrete as is also the roof. The walls above elevation 34 are of brick—12 ins. thick. There is a course of molded brick forming a watertable, and another near the roof level. The cornice is of concrete cast in blocks about 4 ft. long. On top of the cornice is a concrete cap which was cast in place.

The station is divided by a longitudinal wall, into two nearly equal but entirely separate parts, the northerly one being occupied by the sewers, and their accessories such as sluice gates, screens, etc., with their operating devices, while the southerly one contains the pumps, suction and discharge piping, motors, switchboard, etc. The arrangement of the sewers and pumps is shown in plan in Fig. 1. The Park River interceptor, built in 1896, ran directly past the site of the station, its center being about 15 ft. away, while the East Side sewer, 11 ft. further north, was built as far as the station in each direction, a 24-in. tile temporarily carrying sewage across the uncompleted section. In order to have all sluice gates and operating gears housed in the station, loops from these interceptors were brought into this north part of it. Both the Park River interceptor and the East Side sewer are circular in section, the former having a diameter of 4 ft. 8 ins. and the latter 7 ft. at its entrance to the station, enlarging to 7 ft. 6 ins. at the river side down stream of the sluice gate. The flow lines of the two sewers are at elevtion 9.5 and 5.0 respectively. A cross-section of the pumping station is shown in Fig. 2.

In the easterly end of that portion of the East Side sewer which lies within the building is located the large sluice gate. This gate when open provides a clear waterway 7 ft. wide by 5 ft. high at the center and 4 ft. high at the sides, the bottom being the arc of a circle and coincident with the invert of the sewer. Under present conditions this gate is required to keep out river water when it stands between elevations 15 and 33 while the sewage inside is pumped, the level fluctuating between elevations 15 and 8. During low water the gate is open and sewage and storm water flow directly to the river. When, at any future time it is desired to pump the sewage to filter beds, this gate must hold back the sewage while it is pumped during dry weather, with its surface between elevations 7 and 11 and the river between 0 and 11. Any severe rain which occurs during these periods of low water will not be pumped to the lters but will be allowed to flow direct to the iver. To guard against a possible failure o open this gate promptly at such times, here was provided directly above the sluice ate an opening 7 ft. wide by 4 ft. high ith an automatic balanced flap valve which ill allow the entire flow of this sewer to pass ithout backing up higher than elevation 15.0.

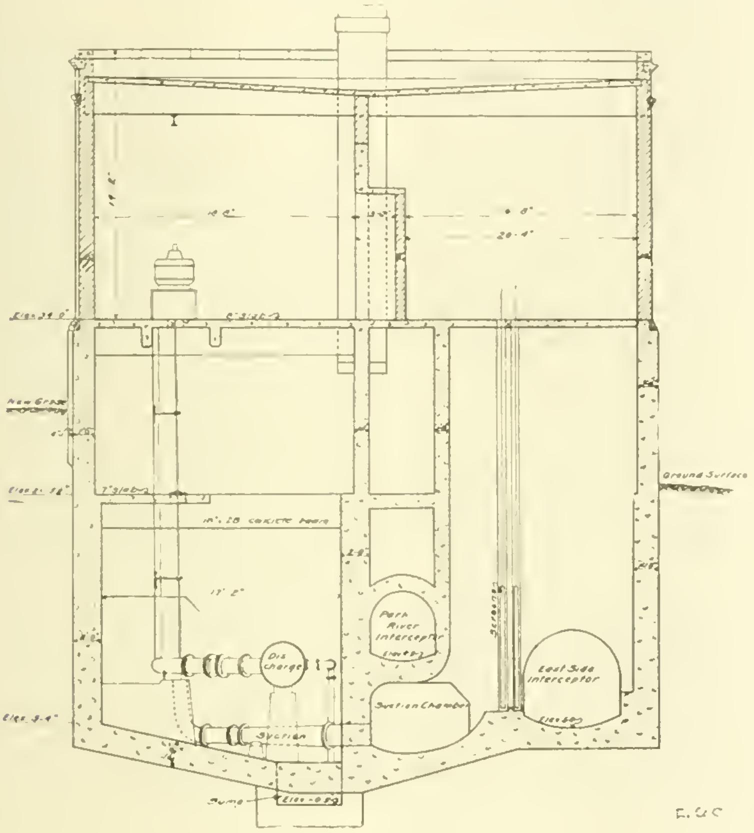


Fig. 2. Cross Section of East Side Pumping Station, Hartford, Conn.

In its course from the sewer to the pumps the sewage will pass through screens to the suction well. There are two openings each provided with two screens, so arranged that one will remain in service while the other is raised and cleaned. Each is made up of a 6-in. channel frame, giving a clear opening 5 ft. wide by 9 ft. high. The opening is traversed by ¾-in. round bars, 2¼ ins. on centers. These screens are raised and lowered by means of a steel cable winding on an overhead drum and operated by chain gear, all of which is located on the main floor. The suction well, located directly beneath the Park River interceptor measures about 23 ft. long, 6 ft. wide and 5 ft. high, its lowest point being at elevation 3.0. Through its south wall, which is the main longitudinal wall mentioned before, pass the suction pipes to each pump.

As the 4 ft. 8 in. sewer has no direct connection with this suction chamber, there is an opening between this sewer and the East Side sewer, located near their entrance to the building, and controlled by a 36-in. sluice gate. There has also been built on this sewer (Park River interceptor) towards the east end of the building a gate chamber which will allow a sluice gate and relief flap valve, similar to the one placed on the other sewer, to be installed at any future time. By these means sewage can be turned from the Park River interceptor to the East Side sewer from whence it will go to the pumps.

At the extreme east end of this part of the building is located the channel into which the discharge pipe from the pumps opens. This channel, which is 6 ft. wide, is open up to the 34 ft. floor and was constructed with a pair of grooves in its walls to allow of a weir being built in order to measure the discharge of the pumps. At its northerly end a 6-ft. circular opening carries the sewage over the Park River interceptor and down into the East Side sewer.

The south half of the station is occupied entirely by the pump room and the motor room, the latter directly above the former. The floor of the pump room, which corresponds to the cellar floor of an ordinary building is a concrete slab 2 ft. thick. Up to elevation 21 ft. 5¼ ins. the walls are of the same thickness, while above that elevation its outside walls are 1 ft. 6 ins. thick and the north division wall is 1 ft. thick. Both the floor slab and walls are reinforced to withstand external water pressure due to high water in the Connecticut River. To help transmit this pressure from the south wall to the north part of the building and also carry a gallery floor at elevation 21 ft. 5¼ ins. there are placed at this level five concrete beams 20 ins. x 28 ins. in section.

There are three 24 in. pumps for handling storm water, each with a capacity of 16,200 gals. per minute against an actual head of 16½ ft.; one 12-in. pump for the normal dry weather flow of sewage, with a capacity of 3,000 gals. per min. against the same head; and a 4-in. sump pump which is able to take care of 260 gals. per minute against a head of 20 ft.

The pump room draws to a central point at which is located the sump 3 ft. 2 ins. deep. Leading from the suction well into this sump is a 6-in. pipe with a gate valve which is ordinarily kept closed. If at any time it is desired to drain the suction well, which action would be necessary if any work were to be done in it, the valve would be opened, and the contents of the well would flow to the sump. While the sump pump was installed primarily to remove any infiltration into the pump room which might occur it will also dispose of this flow from the section well whenever necessary. The 12-in. pump has a branch suction to this sump so that it can be used to empty the pit in case the sump pump fails. A view of the pumps and discharge pipe is shown in Fig. 3.

Each of these five pumping units consist of a vertical shaft centrifugal pump direct connected to an electric motor. The sump pump is set in the sump with its center only 15 ins. above the bottom so that it will run submerged until the pit is nearly drained. The center of each of the other four pumps, its discharge pipe, and the main discharge pipe are at elevation 9.5. As the elevation of the floor varies from 2 ft. 8 ins. to 5 ft. 4 ins., there are two piers to support each pump and five piers to support the main discharge pipe. The suction pipes are also supported by a single pier midway their length.

Each pump has a gate valve on the discharge

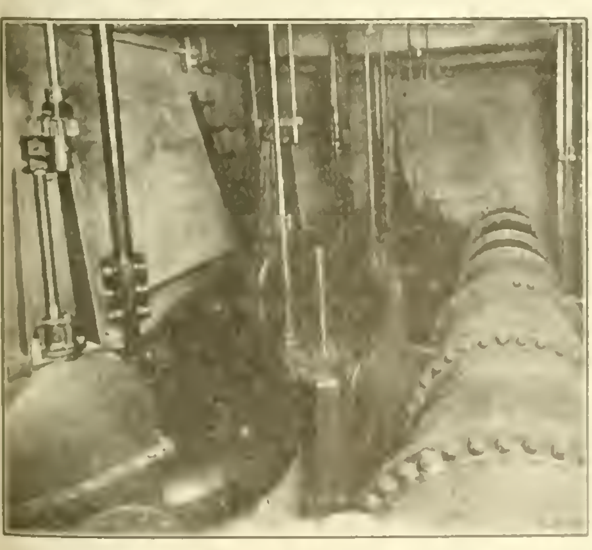


Fig. 3. View of Pumps and Discharge Pipe at Hartford Sewage Pumping Station.

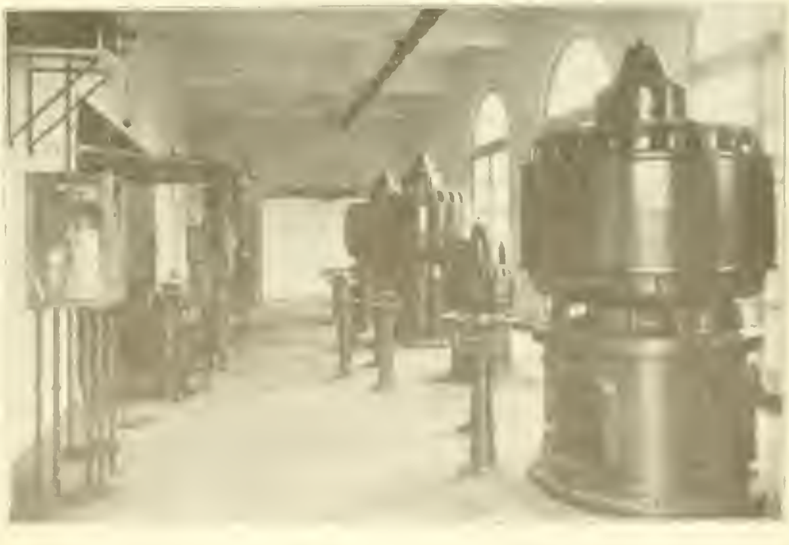


Fig. 4. Interior View of Motor Room, Hartford Sewage Pumping Station.

side and in addition the 12-in. and 4-in. pumps have check valves. The three 24-in. pumps and the 12-in. pumps have gate valves on the suctions. The spindles of all valves, except the 4-in. discharge which terminates at the gallery floor, are brought to the main floor when they are provided with cast iron floor stands having indicators to show the degree of opening of the valve.

Each pump is driven by a vertical, two-phase, four-wire, 60-cycle, induction motor, direct connected to the pump shaft. To allow the thrust bearing of each pump to be placed above the main floor, it was necessary to mount the motor on a base casting about 2 ft. 9 ins. above the floor. The three large motors are rated at 125 HP. each, that for the 12-in. pump at 25 HP. and that for the sump pump at 5 HP. One of 2 HP. operates the 7 ft. x 5 ft. sluice gate. Current is brought into the building at 2,400 volts and the three large motors are run at this voltage while transformers reduce the current supplying the other motors to 230 volts. A view of the motor room is shown in Fig. 4.

The 12-in. sewage pump and the 4-in. sump pump are arranged to start and stop automatically, depending on the water level in the interceptor and sump pit respectively. This, in each case, is accomplished by means of a cylindrical copper float, with chain and counterweight, working in a 6-in. vertical cast iron pipe which is connected to the interceptor in the former instance and the sump in the latter. The chain, connected to a weighted switch, operates this as the float rises and falls. Each of these motors also has hand controlling apparatus. The motor operating the sluice gate is controlled at the gate and in addition has a limit switch which stops the gate at top and bottom of the run if the attendant neglects to do so.

To show the height of water in the East Side sewer and in the Connecticut River at any time, indicating gages have been installed, and in addition, a recording gage to give a continuous ink record of the height of water in the East Side sewer.

As before noted the station will be in operation only a few days each year and therefore while efficiency of operation should be considered, it is not so important as in a station which is operated continuously. It is also very probable that the operator will not be a skilled electrician. For these reasons, the general reliability of the equipment under all conditions, an arrangement of equipment as simple as possible and the installation of protective mechanical and electrical devices were considered as very important.

A complete series of tests on the equipment have not yet been made on account of the fact that the water in the river has not been high enough at any time since the completion of the building. However at that time short tests were made by shutting the gate and allowing the sewage to accumulate and they indicated that all the requirements would be met.

The contractors for the building were the H. Wales Sims Co. of Meriden, Conn., and for all equipment, the Camden Iron Works of Camden, N. J. The consulting engineers were Ford, Buck and Sheldon, Inc., who designed the station, the writer was engineer in charge of construction. All work was under the direction of Mr. R. N. Clark, City Engineer.

Method Employed in Laying a 1,000 ft. Sea-Outlet For a 12-in. Sewer at West Grove, N. J.

A common practice in the disposal of sewage in seacoast towns is to remove the solids from the sewage by means of settling tanks, located at a point near the beach, and then to conduct the tank effluent well out to sea through an outfall sewer. The introduction of the tanks prevents the carrying out to sea of solids which later would be washed back to cause a nuisance at shore. The outfall must be of sufficient length to carry the tank effluent far enough from shore to a depth of water sufficient to afford ample dilution. At such a point the effluent, being lighter than sea water, raises to the surface and mixes with the water. The dissolved oxygen in the water completes the process of purification. This has been found satisfactory at a minimum depth of 20 ft. of water and a minimum distance from shore of 1,000 ft. Strength of structure is requisite in such outfalls since they must withstand the attacks of heavy storms.

A sea outfall of this character was built at West Grove, N. J., during 1913. The methods employed in laying this outfall sewer are here described from information given by Mr. William M. Aitchison, who was the supervising engineer in charge, in an article prepared by him for the Cornell Civil Engineer for February, 1914.

The outfall line was made up of 12-in. galvanized wrought iron pipe manufactured by the Reading Iron Co. The pipe came in lengths averaging 21 ft. and weighing about 48 lbs. per ft. It had threaded joints fitting into recessed couplings.

While the material was being delivered a 3-drum hoisting engine was set up on the beach near the point where the pipe line was to be laid. This being the anchorage of the whole operation great care was taken to secure a substantial setting for it. Eight-foot piles were driven in the sand, cross braced and capped, and the hoisting engine was then bolted to these caps.

For an off-shore anchorage five anchors were set, at a place in a direct line with the proposed outfall, at a distance of 1,325 ft. from shore. This distance was determined with a light line drawn taut. The anchors were set in a line leading away from shore, the two nearest weighing 500 lbs. and the remainder being somewhat lighter. These anchors were fastened together with a steel cable, and an 18-in. sheave attached to the nearest anchor. Through this was run a ⅝-in. steel cable and both ends were brought ashore.

The outlet end of this line was made up with a 45° Ell, looking upward, and a nipple leading into another 45° Ell reversed with respect to the first. This allowed the open end when completed to be 6 ft. from the bottom, so as to be out of danger from filling with sand or becoming otherwise clogged.

A sea-anchor of the contractors' own design, and consisting of two castings and two clamps was fastened to the riser just described. One clamp was bolted over the nipple leading from the lower 45° Ell, and the other over the main pipe just back of the same Ell, both clamps fitting close to the pipe. These were held by eight 1-in. bolts. The sea-anchor weighed 2,200 lbs. in all, and was assembled on the beach just above high water mark.

Starting at the lower Ell the pipes were joined together with the recessed couplings, the threads being painted with white lead to insure watertightness. Platforms were built at intervals of about 30 ft. from this point into the street. Rollers were set on these platforms at such elevations as would give an even fall from the street level and the pipe line was built up on these rollers.

A street leading from the ocean at this point formed a very convenient means to continue the work on the pipe line. The pipes were assembled on this street and were raised on small wooden trucks, the latter being fastened at intervals equivalent to a length and a half of pipe, until the entire length from the river was sufficient to place it 1,200 ft. from the inlet point and have the last pipe above high water mark on the beach. This much completed there was 1,000 ft. of pipe intact, starting at high water mark and extending in shore, all resting on either trucks or rollers.

The sea-anchor was carried on a big truck with two wheels 6 ft. in diameter, made of two thicknesses of 2-in. plank and each having a rim of 2 x 8-in. plank to afford surface bearing.

A hook on the rear of the axle fitted into an eye on the rear of the coupling, thus supporting the sea-anchor and riser, and holding the same free from the ground. Extending out from the axle was a long tongue, at the end of which was a clevice. One end of the steel cable already running through the 18-in. sheave off shore was passed through this clevice and fastened firmly to the sea-anchor, with the other end on a drum of the hoisting engine.

The open end of the riser was next stopped up by means of a wooden plug inserted with white lead. So with the outer end watertight and raised on a truck the entire line was ready and in a movable position.

The hoisting engine was then used to haul the entire length to sea, the weight of the sea-anchor keeping the truck and outer end on the bottom, and the air in the pipe causing the balance of line to be buoyed up. A 3-in. pipe line extending through the entire length of large pipe acted as ballast and held the line partly submerged. This condition removed all of the unnecessary friction, and left only the resistance of the large truck holding the sea-anchor and the small trucks and rollers supporting the pipe. This diminished as the pipe went farther to sea. The small trucks were removed as they came up to the platform supporting the rollers.

The object of the sea-anchor being suspended by this hook and eye method, it will be noted, is to form a convenient means of depositing the pipe in its place when so desired. When the cable line is slackened the weight of the sea-anchor pulls the rear of the axle down, and the tongue up, and at the same time the hook releases the eye, and the sea-anchor is dropped in its permanent place.

The hauling finished, the five anchors off shore were raised singly, starting with the most distant anchor and coming in. This slackened the cable and dropped the sea-anchor. The cable was then cut as close as possible to the arm-anchor, releasing the truck, and with this the off shore work was completed.

The open end in shore, at a point just above high water mark was reduced to a 2-in. bushing, and water was pumped through this into the large pipe until a pressure of about 40 lbs. was reached, thereby blowing the wooden plug out of the riser and sinking the remainder of the pipe in its place.

A block and fall was used in shore to relieve the strain on the five off-shore anchors, one end being fastened to the 12-in. pipe and the other to a second drum on the hoisting engine. It was estimated that a force of about ten tons was required to haul the pipe overboard, this diminishing as the work progressed. Next the in-shore end was lowered to its proper depth and the line continued to the shore manhole.

One of the largest factors to be contended with was the shore current, which tended to bend the pipe out of line before it was lowered to the bottom. This made it imperative that the temporary anchors and the engine base be absolutely secure before the hauling was underway. Another difficulty on such work is the necessity of awaiting the proper weather conditions. While it took but 2½ weeks to put everything in readiness it was a considerably longer time before a combined light westerly wind and low tide made possible its completion.

The total, actual pulling time required was 39½ minutes in the intervals of about 5 minutes each, with lapses between to make sure every part was in proper working order. The total time from start of haul to finish was 2 hours and 15 minutes. At the start of the haul the pipe moved at a rate of 20 ft. in one minute while approaching the end it had increased to 20 ft. in 13 seconds. The average working force was six men, with about six added on the day of the haul overboard.

Matthews Brothers, of Red Bank, New Jersey, were the contractors on this work. Clyde Potts, Civil and Sanitary Engineer, 30 Church St., New York City, looked after the interests of West Grove, the writer being his representative.

Engineering and Contracting

Devoted to the Economics of Civil Engineering Design
and to Methods and Cost of Construction

Volume XLI. CHICAGO, ILL., APRIL 8, 1914. Number 14.

The National Drainage Congress and the Greater Problem of Flood Prevention.

There are now before Congress three bills, all providing in some form for extensive flood control and all stagnating because there is no expressed wish or popular demand for action. No doubt there are contributing reasons for this inaction. For example one of the bills engages the country in a conservation program held by many to be almost chimerical. A second is so apparently sectional as to invite antagonism. The third has perhaps neither of these faults, yet it is almost as completely shelved and will remain so until popular interest is sufficiently intense to express itself to and impress itself upon Congress.

The greater problem of flood control is to arouse an active demand for flood control. This is not an easy task. People forget flood disaster quickly. While recollection is still sharp following disaster they are too pinched to recoup their losses to welcome a demand for further sacrifice to insure their future. As prosperity returns and further immediate disaster does not impend they forget that such insurance is necessary. There are those, of course, who reason better, but they are of two classes. One class comprises the men who form and support our national and local flood prevention associations. The other class comprises, those who see that comprehensive flood control work planned and prosecuted far ahead is necessary for flood control, but who also see that such a work involves certain disturbance of their vested interests and privileges.

The intensity of flood damage is to a great degree due to encroachment by man upon the rights of the waters. He has taken land belonging to the waters for his bridges, his railways, his dams and his multitude of minor works and has given the waters no recompense. Any comprehensive and successful work of flood control necessitates more or less complete return of these usurped rights. Always there must be in such flood control work if not their return, full payment for the rights taken. The spoiler does not contemplate restitution cheerfully. Indeed, in most cases the spoiler of the rights of the waters does not feel that he has taken anything of which restitution should be asked. He considers rather that he has added to improvement and wealth.

The point being made is not the logic or its lack of this reasoning, but the need of overcoming the want of popular interest in flood control which is the result of such reasoning. No active, widespread demand for flood control can come in any other way This brings us to the matter which has been the purpose of this writing. It is the call of the National Drainage Congress for support. In this organization a certain small number of men have given money and much time to work advancing the idea of flood land reclamation in general and Federal financing and direction of flood land reclamation in particular. The material product of their endeavor is a bill to establish a flood protection and drainage fund and to provide for the protection, drainage and reclamation of overflowed and swamp lands. This bill is one of those now before Congress and now, as previously said, being unconsidered because of lack of public interest to urge its consideration.

The National Drainage Congress is not a sectional organization; it is not partisan. Its purpose is to interest people in the desirability of swamp land reclamation and flood prevention. To do this it requires financial and personal support—more money and more members. This support must necessarily come from the engineer and the manufacturer in large part because chiefly they are the ones most able to understand by close knowledge the problem and to appreciate its magnitude and the importance of its solution. These are the readers of this journal, and we frankly urge that they acquaint themselves with the work and purposes of the National Drainage Congress and give it such support as their judgment influences them to give.

Cost Accounting on Construction Work.

Most contractors have what they term a "cost keeping system," although in many cases it receives only such attention as may be given to it at odd times by subordinates. Too often the real value of a comprehensive cost accounting system is not clearly understood. Many contractors are able to determine, approximately, the cost of the completed work, thereby satisfying themselves as to whether there has been a profit or a loss; but few are able to determine, from week to week, the costs of various labor and material items. The latter data are necessary if one is to be able to make such changes in the organization and in the methods used as will insure a reduction in the costs of various items. In order that any comprehensive system of cost accounting be of most value it is essential that men be put in charge of this work who thoroughly understand it and who are not engaged in actual construction work. It must be realized at the start that no comprehensive system will prove successful unless it is followed up closely and is given expert supervision. In the past, many contractors have relied mainly upon their ability to size up the progress and relative costs of different parts of the work after a brief inspection, but such methods are no longer adequate.

In this issue we are giving a detailed description of a cost accounting system in successful use by the Aberthaw Construction Co. of Boston. The system is comprehensive enough to meet the needs of most contractors on building work, and with slight modifications can be made applicable to all classes of construction work. It is the result of careful and continuous study by men whose business it is to do this kind of work, and is a gradual development of methods which have proved successful by actual test. We are glad to publish full details of this system, believing that a careful study of it will convince contractors not only that such a system has great value but that the keeping of it is relatively inexpensive. A standard mnemonic code, which is based upon principles easily learned, has been devised, and this greatly reduces the work of keeping the system, and avoids a confused mass of notes The success of the system lies principally in the fact that the skilled men in charge of it are not engaged in actual construction work, the work being done, principally, in the office. The costs of various labor and material items are computed at the end of each week, which enables those in charge to know definitely what parts of the work are costing too much. Corrective measures can therefore be adopted in time to make an actual saving. The men on the work can find out—in fact are notified—whether or not their work is being efficiently done, which acts as an incentive for economical work.

The systems of cost keeping in use by some merely give the costs of labor items, but a system to be of much value must consider both labor and materials, and must show the relation of each to the costs of various parts of the work. To quote the writer of the article, the purpose of a cost accounting system is threefold: "First, to watch the job from week to week to see if the work is being carried out economically; second, to see whether the cost is above or below that of the original estimate; and third, to furnish information for future use in estimating and in supervising work in progress" Again, "With the change of the times and the change in contractors' methods the attitude of the contractor to cost accounting systems is also changing. The old-time superintendent is giving place to the technical graduate who is a man with engineering training accustomed to view the situation from all sides and relying on actual cost figures rather than on his own judgment to tell whether his work is efficient. Modern competition is becoming so keen and work is taken on such a small margin of profit that it is of vital importance that every item of the work be kept down to its estimated cost, and cost keeping is fast becoming a necessity to all who wish to make a profit out of contracting on construction work." The writer further states that cost accounting on construction work should be handled in connection with the estimating rather than the bookkeeping department, as the estimator should know best what information he requires from a job to check up the work which he has estimated and into what units the costs should be divided. The system described contains the following features which do not appear in most cost accounting systems: first, in addition to the weekly labor costs the average labor cost on each item to date is figured; second, a periodic check on the quantity report is made; third, a standard mnemonic code, revised to suit the requirements of each job is used; fourth, an accounting of material is kept from month to month; fifth, an analysis of the estimate is made for comparison with the weekly costs; sixth, a monthly balance is struck, showing profit or loss to date; and seventh, a field sheet is furnished to the timekeeper, which enables him more regularly and systematically to account for the men on the work.

The system described has been in successful use for some time, and a sufficient number of forms—filled out in detail—have been given to enable one to grasp readily its principal features. The writer of the article and the company with which he is connected deserve credit for an exceptionally well-presented paper on an important and timely subject.

The "Cost of Progress" and the "Profit of Progress" in the Telephone Field.

We may well doubt whether any class of public utility plant averages so short a life as a telephone plant. While this short life is partly due to the extensive use of wooden poles, it is due in much greater degree to that combination of obsolescence and inadequacy now called functional depreciation. Some one has aptly called this sort of depreciation the "cost of progress." The "cost of progress" in the entire electrical industry has been enormous Scarcely has a new device been put into general use before an improved device has rendered the new one "old."

In calculating the cost of rendering a given service we must add liberally for the "cost of progress." There has frequently been criticism of the liberality of such estimates. Somehow too close attention to one side of the ledger is apt to incur forgetfulness of the other side. The profit of progress, as we may term it, always exceeds the "cost of progress," else the engineers and business men in charge of public utilities would not order the frequent renewals and changes in plant. The criterion that an engineer applies is always this: A change in plant is economic when the new plant can be so operated that its annual cost per unit of saleable product, *including therein the interest on the abandoned plant,* is less than the present annual cost.

If this criterion were to lead to writing off nothing from the plant account for abandoned plant, it would result eventually in overloading the plant account to such a degree that new capital entering the same field could produce the most modern plant at much less cost than the capital account of an old company. Hence it is that abandoned plant should be charged off, so as to leave the capital account loaded only with the cost of the modern existing plant. This bookkeeping process, however, should not obscure the fact that the annual saving effected by a new device yields, when divided by the annual interest rate, a sum that more than equals the original cost of the abandoned plant. In short, the profit of progress exceeds the cost of progress. Relatively old as this principle is, repetition of it should not be without value, for there is a tendency to ignore the claims of public utility companies for liberal depreciation reserves.

Often it is argued that although electrical progress in the last 30 years has been very great, there is likelihood that it will be slower hereafter. But before this conclusion is used as a basis for calculating functional depreciation, let two facts be carefully weighed: First, that the use of electricity and other public utility services is increasing much faster than the increase in the population — in general about as the square of the increase in population. Second, that the larger utility companies, as well as the larger manufacturing companies, are maintaining great experimental laboratories manned by expert engineering inventors.

This last mentioned fact is again brought to our notice by a recent statement of Mr. Theodore N. Vail, president of the American Telegraph and Telephone Co. (the Bell System). To quote:

> When the telephone was born, nothing analogous to telephone service as we now know it existed. There was no tradition to guide, no experience to follow.
>
> The system, the apparatus, the methods—an entire new art had to be created. The art of electrical engineering did not exist. The Bell pioneers, recognizing that success depended upon the highest engineering and technical skill, at once organized an experimental and research department, which is now directed by a staff of over 550 engineers and scientists, including former professors, post-graduate students, scientific investigators—the graduates of over 70 universities.
>
> From its foundation the company has continuously developed the art. New improvements in telephones, switchboards, lines, cables, have followed one another with remarkable rapidity.
>
> While each successive type of apparatus to the superficial observer suggested similarity, each step in the evolution marked a decided improvement. These changes, this evolution, have not only been continuous, but are continuing. Substantially all of the plant now in use, including telephones, switchboards, cables and wires, has been constructed, renewed or reconstructed in the past 10 years.
>
> Particularly in switchboards have the changes been so radical that installations costing in the aggregate millions have frequently been discarded after only a few years of use.
>
> Since 1877 there have been introduced 53 types and styles of receivers and 73 types and styles of transmitters. Of the 12,000,000 telephone receivers and transmitters owned by the Bell Company January 1, 1914, none were in use prior to 1902, while the average age is less than five years.
>
> Within 10 years we have expended for construction and reconstruction an amount more than equal to the present book value of our entire plant.

Bearing upon the life of a telephone plant some facts were brought out at a recent hearing before the Public Service Commission of Pennsylvania, in a rate case involving the Bell Telephone system throughout the state of Pennsylvania. Testifying in behalf of the company, Mr. Halbert P. Gillette said that he had made an analysis of the operating expenses of all the Bell telephone plants in Pennsylvania, year by year, for the preceding 30 years. As a result of that analysis he had found that the annual cost of repairs and depreciation had averaged 10.6 per cent of the total capital invested in the plant, including the land and right of way. He also had found that the large amount of capital invested in underground conduits has tended, for some years past, to reduce the repairs and renewals below what is commonly found in many parts of the country.

Speaking of this underground construction as tending to reduce maintenance costs in the future, it must be remembered, however, that each extension of a conduit system causes a heavy functional depreciation of the overhead pole lines that are abandoned at the time the underground lines are put into service. Poles that might have a natural life of 16 years or more, thus come to have a functional life that may not exceed half their natural life.

Referring back to the quotation just given, consider the power for progress that resides in the constant work of a "staff of over 550 engineers and scientists" engaged in experimental and research work for the Bell system. Is it to be wondered that the government-owned telephones of Europe lag far behind this American organization? *It has been said that not a single important telephone invention has been made in Europe in more than a dozen years.* The telephone art and science would have entirely fossilized years ago in Europe had it not been for the progress in America. Those who are now clamoring for government ownership of telephones in America will do well to weigh such facts as these. It is possible, although we doubt it, that by virtue of lower interest charges the government might at first conduct the telephone business at lower cost in this country. But any saving thus effected would be at the far greater loss of what we have termed the profit of progress. Great industrial progress is, we believe, impracticable where the incentive of profit is absent. Better far is it to pay slightly more for a service at present, with every assurance of steady decrease in payments, than to be fettered to a never decreasing payment.

Buildings

Cost Accounting on Construction Work —Detailed Description of an Efficient System.

I

Contractors in general have made some advance in recent years as regards their methods of cost keeping and of analyzing cost data, but the systems used in most cases are not comprehensive enough, nor is enough attention given to them to render them of greatest value. We believe that contractors will adopt better cost keeping and cost analysis systems as soon as they are brought to a realization of the importance of this part of the work. It must be realized at the start that no comprehensive system will be effective unless the contractor is willing to follow it up closely and give it skilled supervision. Contractors should know what various parts of the work are costing in time to enable them to modify inefficient methods, and the system which merely gives the costs of various items of the completed work has little value. The following paper, by Mr. Leslie H. Allen of the Aberthaw Construction Co. of Boston, presented before the Bosotn Society of Civil Engineers, gives in detail the cost accounting system used by this company on construction work. We have largely re-arranged the data contained in the original paper, but shall publish it practically in full. Part II, which will conclude the paper, will be given in the issue of April 22, 1914.

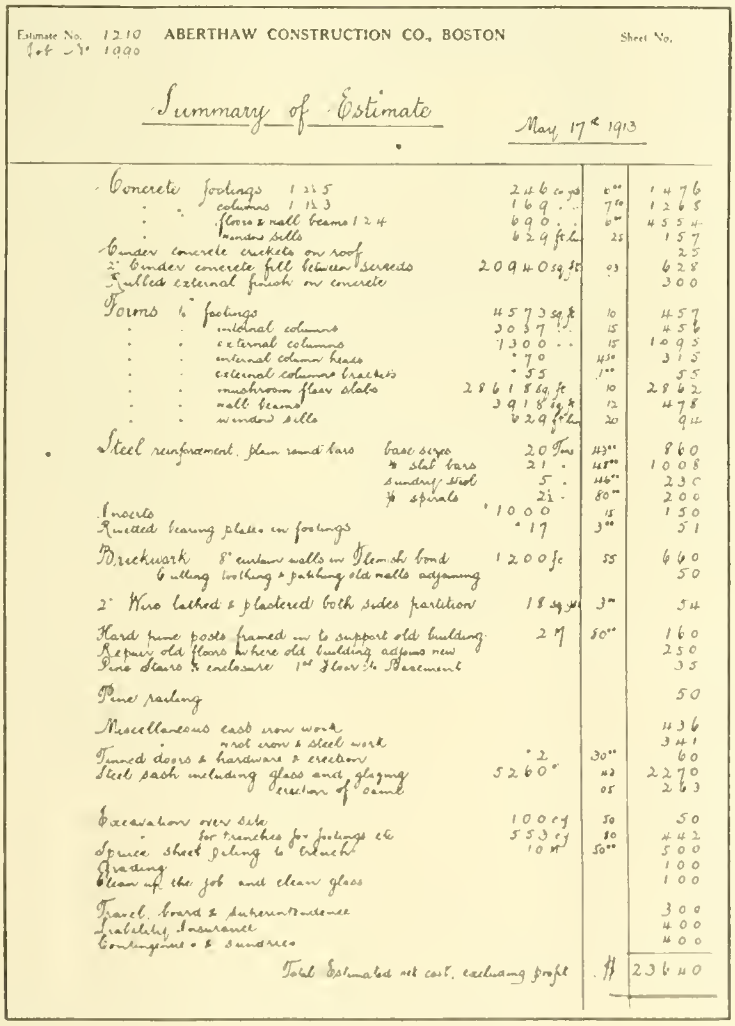
GENERAL.

The problem of cost accounting on construction work is one that has not received the amount of attention and study that its importance warrants. When we consider the vital importance to the contractor on construction work of a knowledge of the cost of his work, it is surprising to find that only a few contractors have succeeded in finding out in detail what the unit costs of their work are. Most of the big commercial enterprises make a point of figuring very carefully their cost of production, but building and engineering contractors seem content to go on in their old ways, with only a hazy idea how their work is coming out, and no definite knowledge as to the amount of profit or loss made until the job is completely finished and paid for. I venture to assert that not more than 10 per cent of the contractors in Boston doing work on a lump sum basis could tell within $1,000 what their profits or losses are on their unfinished contracts now in hand.

It is not hard to find a reason for this state of affairs. The old-time superintendent of construction was usually a mechanic who by reason of special ability had risen to the command of men and become a superintendent. These men, of whom many are still with us (and doing excellent work), are men to whom figures and costs meant very little. They relied for success on their innate common sense and their ability to drive the men under them, and although they made mistakes, some of which would have been self-evident if they had studied costs carefully, yet they did excellent work and achieved results not to be despised even in these days of scientific management, cost accounting and complex organizations of one sort or another. Such men, however, took no interest in cost accounting, and if directed to furnish cost figures took very little care to see if they were made up rightly or not.

Another reason is that there is often a lack of definite instruction given from the office as to how costs are to be kept, and that no man who has studied the problem and knows definitely just what is wanted is in charge of the work and personally responsible for it. The timekeeper on a construction job is a very busy man. Often he has materials to order and check, and many other duties to perform, and has no time to think out the details of a cost accounting system for himself, and he contents himself with as little work in subdivision of payrolls as his experience tells him will "get by." Then when his reports come in they are handled by clerks who do not understand them and who do not make any effort to correct them as they go through, and the cost accounting system becomes unreliable and one is told that "it is impossible to keep accurate costs on construction work."

It is probably true that every contractor and builder has made some attempt to find out the unit costs of the work which he does. Some contractors have got a system that really gives them the information they need. Many firms have a cost keeping system which tells them approximately what their unit labor costs from week to week are, but takes no account of materials and gives them no idea as to how the whole job stands financially. Such men have no real idea as to whether their jobs are profitable or not until the bookkeeper's statement at the close of each job shows the actual profit or loss made. Many have tried to keep up a cost keeping system, but have thrown it up owing to its difficulties and inaccuracies, and rely simply on their bookkeepers to tell them how much expense has been incurred on the job, while their eye tells them how much of the job is done. The writer remembers in his earlier days being directed to visit half-completed jobs for which he had made the estimate and make a survey of same for comparison with the bookkeeper's statements, and this was the only way known by his firm at that time of comparing estimated costs with actual costs to see what profit or loss had been made, although at that time a system of reporting weekly costs was being used similar to that outlined above.



Estimate No. 1210 ABERTHAW CONSTRUCTION CO., BOSTON Sheet No.
Job No. 1990

Summary of Estimate

May 17th 1913

Concrete footings 1:2½:5	246 cu yds	6.00	1476
" columns 1:1½:3	169 "	7.50	1268
" floors & wall beams 1:2:4	690 "	6.60	4554
" window sills	629 ft lin	.25	157
Cinder concrete crickets on roof			25
2" Cinder concrete fill between screeds	20940 sq ft	.03	628
Rubbed external finish on concrete			300
Forms to footings	4573 sq ft	.10	457
" internal columns	3037 "	.15	456
" external columns	7300 "	.15	1095
" internal column heads	70	4.50	315
" external column brackets	55	1.00	55
" mushroom floor slabs	28618 sq ft	.10	2862
" wall beams	3918 sq ft	.12	478
" window sills	629 ft lin	.20	[illegible]
Steel reinforcement, plain round bars base sizes	20 Tons	43.00	860
¼ slab bars	21 "	48.00	1008
sundry steel	5 "	46.00	230
⅜ spirals	2½ "	80.00	200
Inserts	1000	.15	150
Riveted bearing plates in footings	17	3.00	51
Brickwork 8" curtain walls in Flemish bond	1200 ft	.55	660
Cutting, toothing & patching old walls adjoining			50
2" Wire lathed & plastered both sides partition	18 sq yds	3.00	54
Hard pine posts framed in to support old building	2 M	80.00	160
Repair old floors where old building adjoins new			250
Pine Stairs & enclosure 1st Floor to Basement			35
Pine railing			50
Miscellaneous cast iron work			436
" wrot iron & steel work			341
Tinned doors & hardware & erection	2	30.00	60
Steel sash including glass and glazing	5260 ft	.43	2270
erection of same		.05	263
Excavation over site	100 cy	.50	50
" for trenches for footings etc	553 cy	.80	442
Spruce sheet piling to trench	10 M	50.00	500
Grading			100
Clean up the job and clean glass			100
Travel, board & superintendence			300
Liability Insurance			400
Contingencies & sundries			400
Total Estimated net cost, excluding profit		$	23640

Fig. 1. Summary of Estimate.

With the change of the times and the change in contractors' methods, the attitude of the contractor to cost accounting systems is changing, too. The old-time superintendent is giving place to the technical graduate who is a man with engineering training accustomed to view the situation from all sides and relying on actual cost figures rather than on his own judgment to tell whether his work is efficient. Modern competition is becoming so keen and work is taken on such a small margin of profit that it is of vital importance that every item of the work be kept down to its estimated cost, and cost keeping is fast becoming a necessity to all who wish to make a profit out of contracting on construction work.

The purpose of any cost accounting system is threefold: First, to watch the job from week to week to see if the work is being carried out economically; second, to see whether the cost is above or below that of the original estimate; and third, to furnish information for future use in estimating and in supervising work in progress. In other words, first, to determine the items of prime cost and the unit prices of these items; second, to discover what relation these bear to a predetermined selling price; third, to establish new selling prices for future work. The relative importance of these three items is in the order given.

The problem before the contractor's accountant is an entirely different one from that of the bookkeeper or the factory cost keeper. A good deal of the dissatisfaction and incompleteness of many existing systems is because the problem has been approached from the financial point of view rather than the engineer's. I think it was Mr. R. T. Dana who was the first to insist that cost keeping was not a bookkeeper's job and could not be satisfactorily handled by the man who kept the firm's books. This is so. The bookkeeper's viewpoint is the financial one, and deals with totals and balances of cash. The cost accountant's viewpoint is an engineer's, and deals with unit costs and quantities of materials. The two cannot be satisfactorily held by one man unless he has had a thorough training both in bookkeeping and engineering lines. Such a training is rare. The primary object of the cost keeper in a large factory or mill is to determine the selling cost of the articles, and therefore every item of expense burden incurred in carrying on the factory is prorated or apportioned to the cost of the articles produced for sale. The contractor's selling cost is determined beforehand and he is faced with the problem of so splitting up his selling cost that he has a proper appropriation for each item of expense, the very reverse of the factory's accountant's problem. It is for this reason that I have placed prime cost first as being the most important, and selling cost last as of least importance.

The estimate of cost on which a contract is taken is more like the budget appropriation of our government, which having a predetermined amount of money to spend proceeds to allot as much as is possible to each government department in proportion to its needs, the total of such appropriations being equal to the amount of the estimate of cost.

The contractor's cost accounts, if they are to be of any use, must show not only the amount of money spent in the work, but the way in which it has been spent; and this can not be shown if items of general expense, such as plant, watchmen, etc., are all distributed among the items of excavation or concrete. The result may be financially accurate but uninforming to the contractor. For instance, on the cost accounts on the Panama Canal which appear in the *Canal Record* it will be noted that the cost of plant, track forms and general expense, etc., all are worked out in terms of per cubic yard of concrete. This is correct from the financial viewpoint, and the result shows the cost of the concrete to the government. But it does not tell an engineer whether the work is being done efficiently or not. He wants to know the cost of setting up and repairing the plant, the cost of handling the material, the detailed cost of form work, while general expense is meaningless to him unless he knows the details. When the cost of moving tracks, repairing plant, etc., is reported at so much per cubic yard as a subdivision of excavating cost, the cost of the operation cannot be criticised. It is very necessary that this distinction be kept in mind and that costs should be worked out from an engineer's standpoint rather than from a bookkeeper's or an accountant's standpoint. The information turned out by the cost-accounting department should not only show results but should give information showing the reason why the results are so.

In the writer's judgment, the cost accounting on construction work should be handled in connection with the estimating department rather than in connection with the bookkeeping department. The estimator should know best what information he requires from a job to check up the work he has estimated, and into what units the costs should be divided. He is then enabled to keep closely in touch with actual work in progress and to compare it with his estimate from time to time.

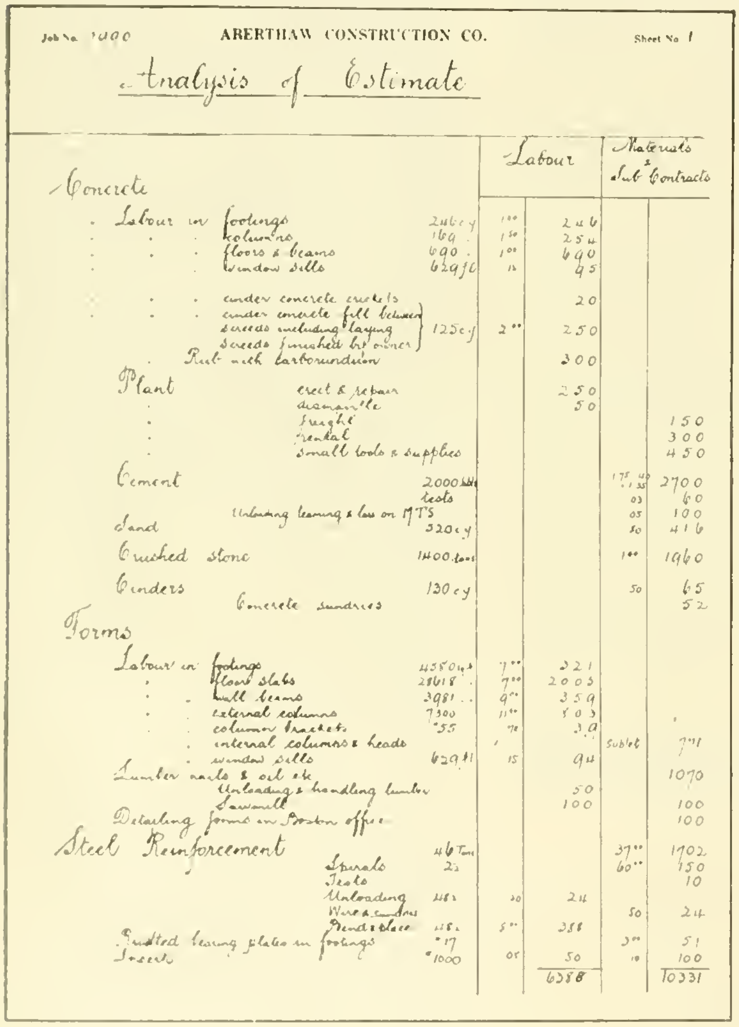
The method usually employed by contractors in their cost accounting is to divide up the time spent on the job under certain fixed headings, such as excavation, brick work, mason work, concrete work, carpentry, and so on, and to make a report of the quantity of each kind of work executed, and to work out from same either daily or weekly the unit labor costs; at the close of the job, to work these up into totals and also to work up at this time the cost of all materials into units and combine them with the labor costs, giving the total cost of labor and material on each item of the work. This works well in some offices as far as it goes, and shows whether each week's work was efficient and economical. But if no account is kept of material until the job is closed, and no check is taken on quantities reported by the men on the job, and no comparison is made with the estimate, its value is not very great. In the system used by the Aberthaw Construction Co., we have added from time to time the following features which do not appear in most accounting systems. First, in addition to the weekly labor costs, the average labor cost on each item to date is figured. Second, a periodical check on the quantity report is made. Third, we use a standard mnemonic code revised to suit the requirements of each job. Fourth, an accounting of material is kept up from month to month. Fifth, an analysis of the estimate is made for comparison with the weekly costs. Sixth, a monthly balance is struck, showing profit or loss to date. Seventh, a "field sheet" is furnished to the timekeeper, which enables him to keep track more regularly and systematically of the men at work.

It is the purpose of this paper to describe the Aberthaw company's system in detail, taking up in logical order the above points and matters incidental thereto. The present system was not invented in a day but has been a gradual development. The cost keeping system was installed early in the history of the company by Mr. Wason, the president of the company, who laid work out upon the usual lines adopted by other contractors as indicated above. The above additional features have been added since the beginning of 1910 in the order indicated above, and the complete system may be said to have been in working order for over two years, long enough for us properly to try it out and to know that it really gives us the information we need. In other words, the system I am about to describe is not the dream of a theorist, but something that is really practicable; that is not expensive in operation and that any contractor can use if he has but the patience to study his problem and insist on getting just the results he wants.

Although the writer's firm specializes in one branch of construction work the system is equally applicable to any other work of similar nature, whether in the line of heavy engineering or the building of modern office buildings or frame dwelling houses.

Section I.—The Analysis.

At the start of a job, an estimate is made of its cost in the usual way. This estimate is then analyzed to show its component costs. Figure 1 shows an estimate on a small job executed by the Aberthaw company this year. Figures 2 and 3 show the analysis of the same. This analysis is almost self-explanatory, but it will be well to call attention to one or two points in it. It will be noted that the 7 items of concrete and finish in the estimate have been resolved into 19 items in the analysis, and the 7 items of forms in the estimate have been resolved into 11 items in the analysis, and so on, each item being resolved into its component parts, and where these parts are alike in two items adding them together. For instance, concrete labor in footings appears by itself, but the cement, sand and stone are added to those in the columns and floors. By adding together, it will be found that the 7 items of concrete in the estimate have the same total cost as the 19 items in the analysis.

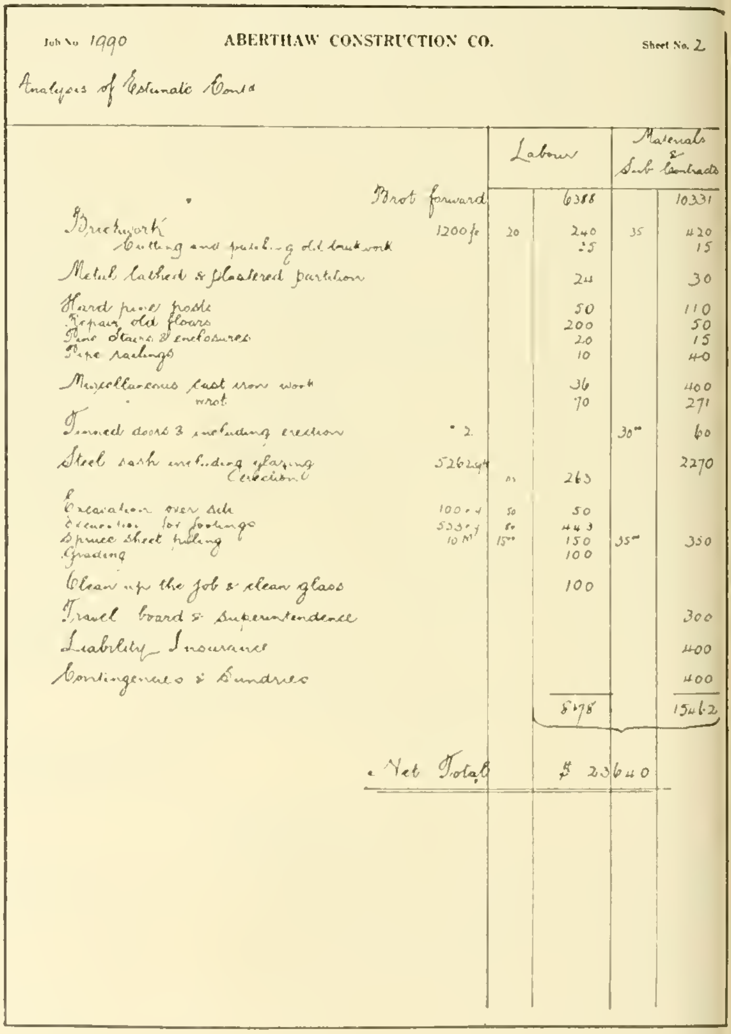


Job No. 1990 **ABERTHAW CONSTRUCTION CO.** Sheet No. 1

Analysis of Estimate

		Labour		Materials & Sub Contracts	
Concrete					
Labour in footings	246 c.y.	1.00	246		
" " columns	169 "	1.50	254		
" " floors & beams	690 "	1.00	690		
" " window sills	629 f.t.	.15	95		
" " under concrete cricklets			20		
" " under concrete fill between screeds including laying screeds finished by owner	125 c.y.	2.00	250		
" Rub with carborundum			300		
Plant erect & repair			250		
" dismantle			50		
" freight					150
" rental					300
" small tools & supplies					450
Cement	2000 bbl			1.35	2700
tests				.03	60
Unloading teaming & loss on MTS				.05	100
Sand	520 c.y.			.80	416
Crushed stone	1400 tons			1.40	1960
Cinders	130 c.y.			.50	65
Concrete sundries					52
Forms					
Labour in footings	4580 sq ft	7	321		
" floor slabs	28618 "	7	2003		
" " wall beams	3981 "	9	359		
" " external columns	7300	11	803		
" " column brackets	55	70	39		
" internal columns & heads				sublet	741
" window sills	629 ft	15	94		
Lumber nails & oil etc					1070
Unloading & handling lumber			50		
Sawmill			100		100
Detailing forms in Boston office					100
Steel Reinforcement	46 Tons			37.00	1702
Spirals	2½			60.00	150
Tests					10
Unloading	48 t	50	24		
Wire & clamps				50	24
Bend & place	48 t	5.00	388		
Slotted bearing plates in footings	17			3.00	51
Inserts	1000	05	50	10	100
			6388		10331

Fig. 2. Analysis of Estimate (Continued in Fig. 3).



Job No. 1990 **ABERTHAW CONSTRUCTION CO.** Sheet No. 2

Analysis of Estimate Cont'd

		Labour		Materials & Sub Contracts	
Brot forward			6388		10331
Brickwork	1200 ft	20	240	35	420
Cutting and patching old brickwork			25		15
Metal lathed & plastered partition			24		30
Hard pine posts			50		110
Repair old floors			200		50
Pine stairs & enclosures			20		15
Pipe railings			10		40
Miscellaneous cast iron work			36		400
" wrot			70		271
Tinned doors 3 including erection	2			30.00	60
Steel sash including glazing	5262 sq ft				2270
Erection		05	263		
Excavation over site	100 c.y.	50	50		
Excavation for footings	553 c.y.	80	443		
Spruce sheet piling	10 M	15.00	150	35.00	350
Grading			100		
Clean up the job & clean glass			100		
Travel board & superintendence					300
Liability Insurance					400
Contingencies & Sundries					400
			8178		15462
Net Total			$ 23640		

Fig. 3. Analysis of Estimate (Continued from Fig. 2).

This analysis is used as a basis of comparison during the life of the job, and the original estimate is not referred to at all. This is different from the general practice, as it is usual to build up the units of labor costs on a job in a synthetical manner to compare with the estimate, rather than to analyze the estimate to compare with the units on the job. This is a very important feature of the system and is a solution of one great difficulty. The usual method is to try and build up the price of each item from the information on unit costs given, so as to compare it with this item in the estimate. Take, for instance, the item of concrete. The price of concrete is compiled by reckoning up the labor of unloading cement, sand and stone and the cost of these materials, and also the labor of mixing and placing and finishing the concrete and the cost of the tools and plant. (I do not mention forms, which I regard as an entirely distinct and different item.) If at the end of a month 500 cu. yds. of concrete have been placed and material enough to mix another 600 cu. yds. is on the ground and all the plant is set up, it will be a very difficult matter to determine exactly how much labor and material should be charged to work which is done and how much is chargeable to future work. If all the mason's staging is erected and only one-third the brickwork is done, the cost of brickwork will be unbalanced in the same way. To solve this difficulty we use an analysis of the estimate and keep the costs on each item of the analysis separately. The estimate is put away and never referred to again, and all comparisons are made with the analysis. You will see when I describe the monthly statement how easily this takes care of difficulties like those referred to, or to unfinished work. It is not until the job is completed and the books are closed that I make a synthetical summary showing what a cubic yard of concrete or a cubic foot of brickwork has cost.

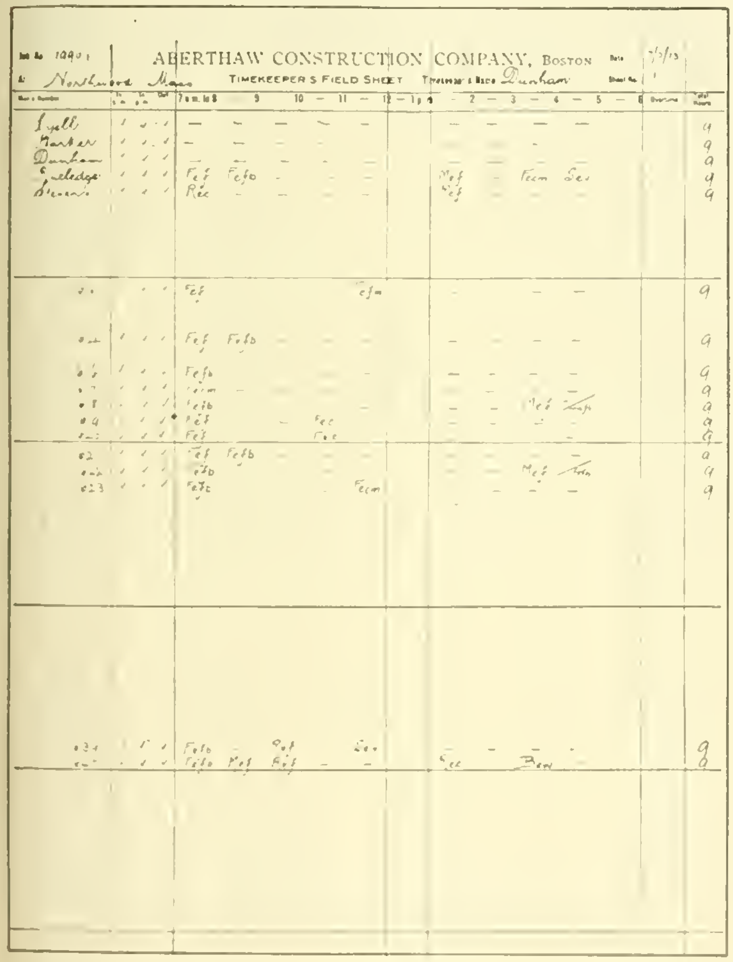
I lay stress on this because the difference between analysis and synthesis in accounting is vital, and it is my belief that it is not possible to get a clear and accurate idea of the fluctuations of cost on construction work by a synthetical method, because labor and material and overhead expense are so distributed that they cannot be properly identified or criticised. It will be noticed that labor and material are kept in separate columns in the analysis and are kept entirely distinct all through the job. For accounting purposes, material includes sub-contracts, insurance, traveling expenses, electric power, etc., and in fact everything except labor and teams. Teams when hired by the day are reckoned as labor, but teams working under contract, at an agreed price per load, per yard, etc., are reckoned as sub-contracts and kept in the material account.

On the work of most contractors it will be found that the amount and cost of the materials will not vary very much, and, except for checking consumption of cement, coal and lumber, the profit or loss made on materials at the start of a job will remain steady all through. But the labor does fluctuate exceedingly from week to week. It is on the labor side that most of the losses or profits may be expected. In this paper and in the writer's firm a good deal more attention is given to the labor side than to the materials. At the same time the material must not be overlooked, as wasteful use of cement, lumber, etc., may run the cost of a job up unexpectedly, to say nothing of the need of a periodical check, if there is any suspicion of graft among subordinates.

We furnish a copy of the analysis in notebook form to the superintendent on the job as well as to our general superintendent and to the heads of the firm. This is particularly useful to the job superintendent, as he knows what the office expects him to accomplish in the way of costs. If his judgment on costs is not sound, it tells him what the items of his work ought to cost.

Section II.—The Code.

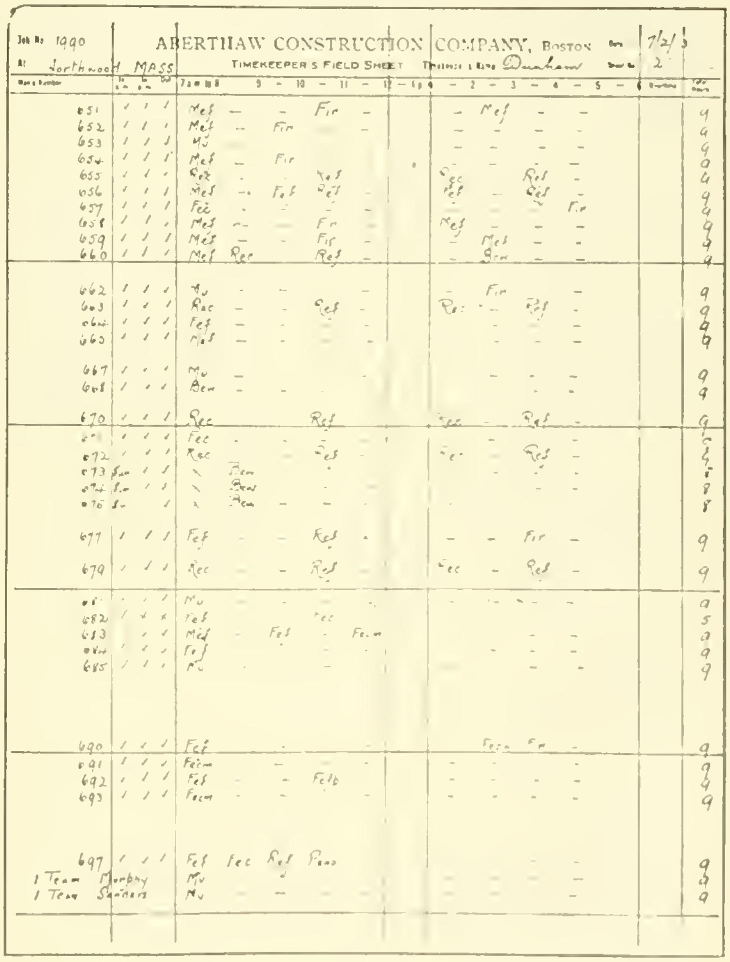
Having analyzed the estimate, instructions are made out for the timekeeper in the form of a code, which is shown in Table I. This code is made up from the standard mnemonic code used by the author. The code differs from that used by other contractors only in the fact that the mnemonic principle is used instead of numbers, and that the divisions of time have been carried a good deal further than is usual. Although this does not seem to be an important item in itself, it is so because it simplifies the work of the timekeepers on the job to a very large extent, and insures more accurate and intelligent reports being made from the job.



Job No. 1990 ABERTHAW CONSTRUCTION COMPANY, Boston Date 7/2/13

At Northboro Mass TIMEKEEPER'S FIELD SHEET Timekeeper's Name Dunham Sheet No. 1

Fig. 4. Timekeeper's Field Sheet (Continued in Fig. 5.)



Job No. 1990 ABERTHAW CONSTRUCTION COMPANY, Boston Date 7/2/13

At Northboro MASS TIMEKEEPER'S FIELD SHEET Timekeeper's Name Dunham Sheet No. 2

Fig. 5. Timekeeper's Field Sheet (Continued from Fig 4.)

TABLE I—STANDARD TIMEKEEPING CODE

Main Divisions—Kinds of Work

Initial Letter

P	Plant
D	Digging, earthwork and rockwork, and items in connection
M	Concrete
F	Forms.
R	Reinforcement
K	Finish of concrete surfaces.
C	Finish carpentry (windows, flooring, etc.) and any carpentry not belonging to P, D or F
S	Miscellaneous steel and iron work and other metal work.
B	Brick masonry, stonework, tile, Akron pipe, etc.
Z	Miscellaneous.
X	Extra work (prefix to any of above).

Subdivisions—Kinds of Labor.

Second Letter (for all main divisions except D and K):

a	Making or preparing, viz., making up forms, mixing concrete, bending or fabricating steel, etc.
e	Erecting, placing or building, viz., erecting forms, placing concrete, laying brick, fixing sash, etc.
i	Removing, stripping or cutting away, viz., stripping forms, cutting away concrete or brick, etc.
o	Repairing, viz., patching voids in concrete, repairing mixer, etc.
u	Receiving, unloading, piling, loading, etc., viz., receiving cement, sand, lumber, etc., unloading and storing same, unloading plant and reloading at close of job, etc.

Second Letter with D:

a	Excavate.
e	Backfill.
i	Pumping.
o	Grading
u	Drilling and blasting

Second Letter with K

a	Picking.
e	Plastering.
i	Rubbing with carborundum.
o	Repairing, filling voids, cleaning floors, etc.
u	Granolithic finish laid integral with the slab
[illegible]	Granolithic finish laid after concrete has set
[illegible]	Cement wash

Subdivisions—Location of Work in the Building

Third Letter (for all main divisions except D, S and C)

b	
bc	Belt course
c	Columns
ch	Column heads (mushroom)
cc	Cinder concrete
d	Footings
dp	Drain pipe (tile)
f	Floors
fs	Floor slabs.
fb	Floor beams (beam construction)
fm	Corrugated metal to slabs
g	
h	Cellar or basement.
j	
k	Cornice
l	Lumber
m	Monitor or pent house
n	Lintels
p	Paving or sidewalk.
q	
r	Rubbish.
s	Stairs.
t	
v	Vault lights.
w	Walls.
rw	Retaining walls.
ws	Window sill.
cw	Curtain walls.
x	Cement.
y	Sand.
z	Stone or gravel.

Third Letter with C:

c	Column.
d	Door.
df	Door frame.
dt	Door trim.
f	Floor.
fb	Floor beam.
fs	Floor screed.
fp	Sub-floor plank.
ft	Top floor maple.
g	Gates.
h	Hardware.
ho	Operating gear.
j	
k	Cornice.
l	Lumber
m	Monitor or pent house
ms	Monitor sash
n	Lintels
p	Partition
pl	Platform
r	Roof
rb	Roof beam
rp	Roof plank
rt	Roof truss
[illegible]	Stairs
t	
[illegible]	
[illegible]	
[illegible]	
[illegible]	Miscellaneous

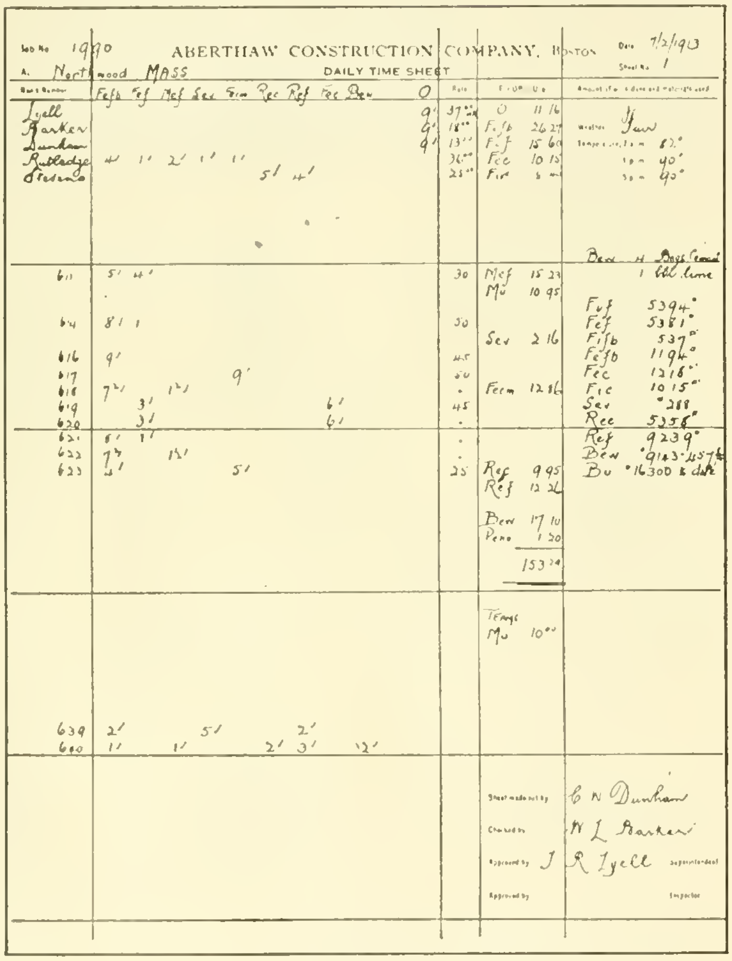
Third Letter with S

b	I-beams, channels, etc.
c	Columns
cb	Column bases
d	Doors
[illegible]	Door [illegible]
df	Door frames
ds	Door sills
f	Fl[illegible]
g	
h	
[illegible]	Bolts
[illegible]	Cornice
l	
m	
n	Lintels

p	Pipe.
q	
r	Railings, gratings, etc.
s	Stairs.
sr	Stair rails.
sl	Sleeves.
t	Trusses.
v	Inserts for sprinklers, etc.
w	Windows.
ws	Window shutters.
wg	Window guards.
x	
y	
z	Miscellaneous.

Third Letter with P:

b	Boiler.
c	Crusher and elevator.
ch	Chute.
d	Derrick.
f	
g	Locomotive, portable engine and boiler.
h	Hoisting engine.
j	
k	
l	Elevator tower, bucket and hoist.
m	Mixer and engine.
n	Temporary buildings.
p	Pump.
q	
r	Runways, staging, ladders and guard rails, etc.
s	Wood-working shop, saw bench, planes, etc.
t	Track.
v	
w	Water supply.
x	
y	
z	Miscellaneous.


Job No 1990 ABERTHAW CONSTRUCTION COMPANY, BOSTON Date 7/2/913
At Northwood MASS DAILY TIME SHEET Sheet No 1
Man's Number | Fefb Fef Mef Sec Sem Rec Ref Fec Bew O | Rate | | Amount of work done and material used

Lyell 9' | 37'' | O 11 16
Barker 9' | 18'' | Fifb 26 27
Dunham 9' | 13'' | Fef 15 60
Rutledge 4' 1' 2' 1' 1' | 36'' | Fec 10 15
Stevens 5' 4' | 25'' | Fir 5

Weather Fair
Temperature 7 a.m. 82°
1 p.m. 90°
5 p.m. 90°

Bew 4 Bags Cement
1 bbl lime

611 5' 4' | 30 | Mef 15 23
Mu 10 95
614 8' 1 | 30
Sev 2 16
616 9' | 45
617 9' | 50
618 7½' 1½' | . | Ferm 12 16
619 3' 6' | 45
620 3' 6' | .
621 8' 1'
622 7½ 1½'
623 4' 5' | 25 | Ref 9 95
Ref 12 26
Bew 17 10
Peno 1 20
153 24

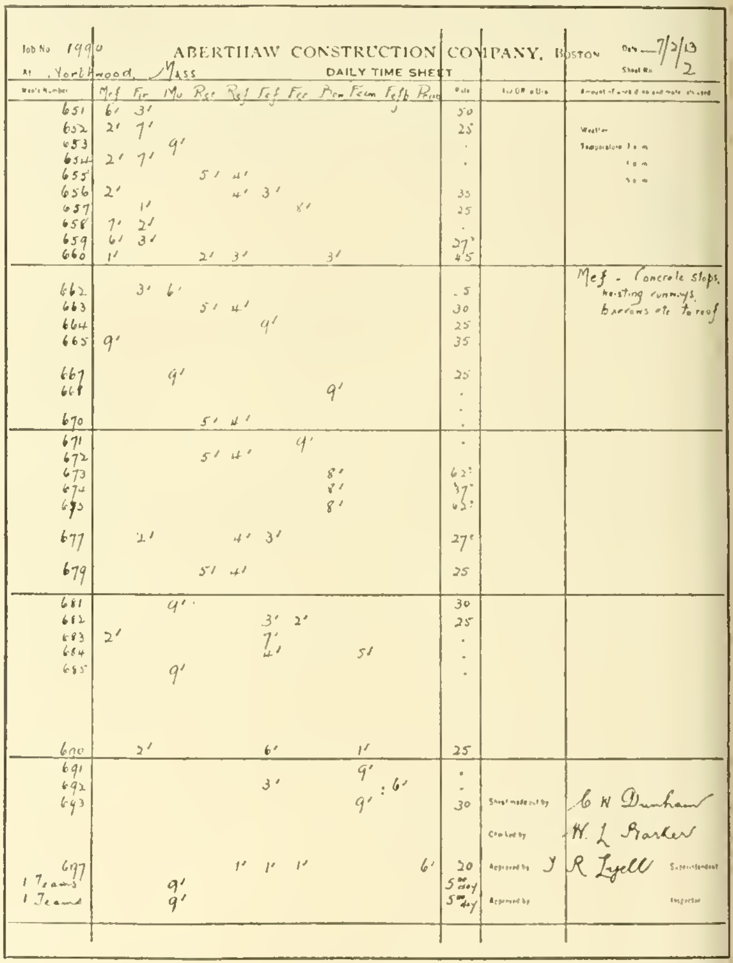
Fuf 5394°
Fef 5381°
Fifb 537°
Fefb 1194°
Fec 1216°
Fic 1015°
Sev 288
Rec 5356°
Ref 9239°
Bew 9143 · 457½
Bu 16300 & date

Temps
Mu 10''

639 2' 5' 2'
640 1' 1' 2' 3' 2'

Sheet made out by C N Dunham
Checked by W L Barker
Approved by J R Lyell Superintendent
Approved by Inspector

Fig. 6. Daily Time Sheet (Continued in Fig. 7).


Job No 1990 ABERTHAW CONSTRUCTION COMPANY, BOSTON Date 7/2/13
At Northwood, Mass DAILY TIME SHEET Sheet No 2
Man's Number | Mef Fir Mu Rec Ref Fef Fec Bew Fem Fefb Peno | Rate | | Amount of work done and material used

651 6' 3' | 50
652 2' 7' | 25
653 9' | .
654 2' 7' | .
655 5' 4'
656 2' 4' 3' | 35
657 1' 8' | 25
658 7' 2' | .
659 6' 3' | 27½
660 1' 2' 3' 3' | 45

Weather
Temperature 7 a.m.
1 p.m.
5 p.m.

662 3' 6' | .5
663 5' 4' | 30
664 9' | 25
665 9' | 35
667 9' | 25
668 9' | .
670 5' 4' | .

Mef - Concrete stops, hoisting runways, barrows etc to roof

671 9' | .
672 5' 4'
673 8' | 62½
674 8' | 37½
675 8' | 62½
677 2' 4' 3' | 27½
679 5' 4' | 25
681 9' | 30
682 3' 2' | 25
683 2' 7' | .
684 4' 5' | .
685 9' | .
690 2' 6' 1' | 25
691 9' | .
692 3' 6' | .
693 9' | 30
697 1' 1' 1' 6' | 20
1 Teams 9' | 5 day
1 Teams 9' | 5 day

Sheet made out by C N Dunham
Checked by W. L Barker
Approved by J R Lyell Superintendent
Approved by Inspector

Fig. 7. Daily Time Sheet (Continued from Fig 6).

All contractors who have attempted any cost keeping will agree on the necessity for some sort of a code to report work done, not on the ground of secrecy but to obtain concise, quick descriptions. It has been found that if timekeepers are simply told to report the work done and describe it, their descriptions will be misleading and verbose, and in going their rounds they will probably make up a code for themselves which afterwards they have to turn into a written description.

It may be worth while to spend a little time in explaining the principles of the mnemonic code used. The initial letter is always a capital and indicates the kind of work to be done. For instance, M stands for concrete Masonry, F for Forms, R for Reinforcing Steel, S for Structural Steel and miscellaneous iron work, B for Brickwork, and so on. As far as possible the initial letters chosen are mnemonic, that is, they are the first letters of the items they represent. The second letter is always a vowel and indicates what is being done in the division of work which it is describing: "a" stands for making items before setting, and "e" stands for erecting or setting or fixing in place; "i" stands for stripping or removing or pulling down; "o" stands for repairing or patching; "u" is a general utility item standing for unloading and other similar items. It will be seen that these are mnemonic in that each vowel is the vowel sound of the word that it represents. The third letter, which is always a consonant, indicates mnemonically the place or part of the building in which the work is done; that is, "f" indicates floors, "c" columns, "b" beams, "k" cornice, "l" lumber, "d" doors, and so on. As far as possible these are mnemonic, but it is not possible to make every item so, and some consonants have to be arbitrarily chosen to make out. As the standard code is the same on every job and is used simply with adaptations to meet the requirements of the work in hand, these arbitrary symbols are quickly learned. It will be noticed that the third letter has to be a different one in the case of items relating to plant.

It should be an easy matter to revise this code to apply to any other sort of construction work, or to industrial work of any kind, bearing in mind the general principles on which it is framed.

Although at first sight it looks complicated, yet it has been found to be very simple in practice, and new timekeepers and superintendents very quickly pick up its essentials.

It has the advantage that if any work is done on the job which was not known of or contemplated when the original code was made out, it is a simple matter to adapt the letters and make up a fresh code word for the new work. As a matter of precaution, we always require our timekeepers to give the explanation of a new code word whenever he makes one up, although very often we can read them without such explanation, owing to our familiarity with the principles on which the code is based.

From the standard code as given above a special code is written out in the head office for the job before the work is started, and from time to time additions are made from the office or by the job, to meet special items of work met with on the job; but all the main items of work, such as concrete in columns and floors, forms and reinforcement, have the same code word at all times, for example:

M—Concrete.

	Mix.	Place.	Fill voids, smooth up, etc.
Footings	Mad	Med	Mod
Floors and roof..	Maf	Mef	Mof
Columns	Mac	Mec	Moc
Walls	Maw	Mew	Mow

F—Forms.

	Make.	Erect.	Strip.
Footings	Fad	Fed	Fid
Floors and roof..	Faf	Fef	Fif
Columns	Fac	Fec	Fic
Walls	Faw	Few	Fiw

Section III.—The Timekeeping.

(A) THE FIELD SHEET.

On the job the timekeeper is supplied wit what we call a "field sheet," an example o which is shown in Figs. 4 and 5. This fiel sheet is, we believe, an innovation in timekeeping. A fresh sheet or sheets is use every day and is carried around by the timekeeper on the job. In the left-hand colum he enters the numbers of all the men who ar at work, and it will be noticed that the tent line is a heavy line and that numbers 10, 20 30, and so on appear on these heavy lines always, and if necessary, blanks are left in be tween to insure this being done. (The object of this will be explained later.) Afte having checked the men in, in the second column from the left, the timekeeper starts o his rounds and in the column headed 7 to he places against each man's number the cod word for the work he is doing. On makin his second round, which is usually about 1 o'clock, if the man is still on the same wor he simply places a check in the intervening columns to show that the same work is going on. The timekeeper has to make at least four rounds every day and has to find each man on every round. If a man is not found, his pay is docked unless he can satisfactorily explain where he was.

It will be noted that the field sheet provides columns for checking the men in after the noon hour and for checking men out at night and for overtime. The last column is the total number of hours worked during the day, which is used in making up the payroll, as will be shown later.

In going his rounds, the timekeeper carries the field sheet in a stiff binder similar to that used by the express companies. On a large job, sometimes as many as six or eight sheets are used every day. They are, of course, all numbered up with the numbers of the men before he starts on his rounds. The advantages of this field sheet are as follows:

(1) The timekeeper keeps a permanent record of what the men are doing every hour in the day. This is an improvement on the old method, when timekeepers used to take the time on old pieces of scrap paper, backs of envelopes, or pieces of old board, and kept no permanent record of the work in any form. In that way men were frequently missed and their time afterwards guessed at, and there was no check on the men that showed they were all at work.

(2) The superintendent can easily check the work of a timekeeper. Most of our superintendents, each time that they go out on to the work, make a note of the numbers of three or four of the men and note what they are doing at that time, and on coming in refer to the timekeeper's field sheet to see what work he has entered them as being on, and by picking out at random a few men every day in this way it is easy to find out whether the timekeeper is doing his work correctly.

(3) It is easy to disprove men's claims for more time than they are allowed, as in addition to the record on the payroll we have this actual check on what the men were doing hour by hour, and a man who claims his time is short is confronted with the field sheet, which shows exactly what he has been doing.

(4) On a large job, two men can work together on the timekeeping, for on a job where the payroll is as much as $4,000 a week it is all one man can do to go round the work and note the men's time. He turns in his field sheet to an assistant timekeeper, who makes out the payroll and time sheets from it. Under the old methods, when rough notes were made on scrap paper, this would be a very difficult if not an impossible thing to get done correctly, and the result would be that the timekeeper's field work would suffer and the cost accounts would be inaccurate.

There are necessary limits to the number of subdivisions made in the work. Some timekeepers with more zeal than discretion will multiply subdivisions without end if not watched. It is a standing instruction to our men that the laborer who shakes out and bundles cement bags is to be charged to the largest concrete item of the day, and that the saw filer is to be reported on the largest piece of form work. All time on temporary dams for construction joints in concrete floors are reported with floors. The superintendent, timekeeper and water boy are not so charged, but are prorated on every item, as will be seen in Section IV (A).

(B) THE TIME SHEET.

Figures 6 and 7 show the daily time sheet. In this it will be noted that the ruling is the same as on the field sheet, with every tenth line reserved for numbers 10, 20, 30, and so on, so that the men's numbers appear always on exactly the same line on each sheet. In the left hand column are the numbers of the men and on the top of the succeeding column are written the code words of the work in progress. Under these is entered against every man the number of hours he has spent on the section of the work denoted, and on the right-hand side is a column for the man's rate. The next two columns are headed "For Office Use," and when the sheets come into the office the total amount spent on each subdivision of the work is worked up and entered there, as is shown in the example. The extreme right-hand column is reserved for the report of the quantities of the work done, which are entered by the timekeeper every day, or, in some cases where the work is unfinished at the end of the day, every second or third day.

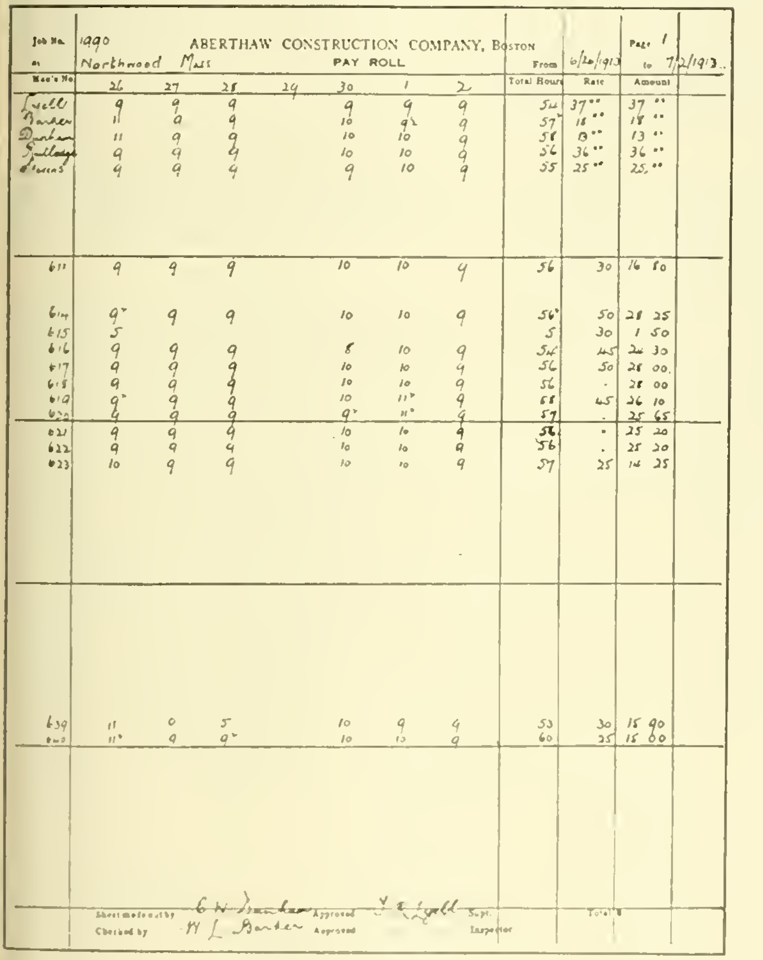
These sheets are entered up by the timekeeper on the job, sometimes in the evening after the day's work is done, but usually in the morning following the day in which the work was done. They are mailed to the office daily.

(C) THE PAYROLL.

Figures 8 and 9 show the payroll, which is also ruled up in sets of ten lines, like the field sheet and the time sheet. This is similar to all contractors' payrolls and needs no comment, except to point out the immense saving of time that has been made by the use of time sheets, field sheets and payrolls, all ruled uniformly, and insisting on leaving blank spaces rather than using every line on the time sheets and payroll. The payroll is filled in simply by placing the field sheet over the payroll, and transferring the number of hours' work without even referring to the men's number. The payroll is then checked by the time sheet, so that there is a circular check on the whole of the operation.

(D) THE QUANTITY REPORT.

This has already been referred to when considering the time sheet. It is of course of vital importance that the quantities should be accurately reported. We make it the duty of the engineer in charge of the level and transit to compute the quantity of work done each day and turn it in to the timekeeper to enter in the column provided. It is often a difficult matter to insure correct reports being received on the quantities of work done, and at the time the system was installed the writer made a practice of visiting jobs monthly and making a rough survey of them where-



Job No. 1990 ABERTHAW CONSTRUCTION COMPANY, Boston Page 1

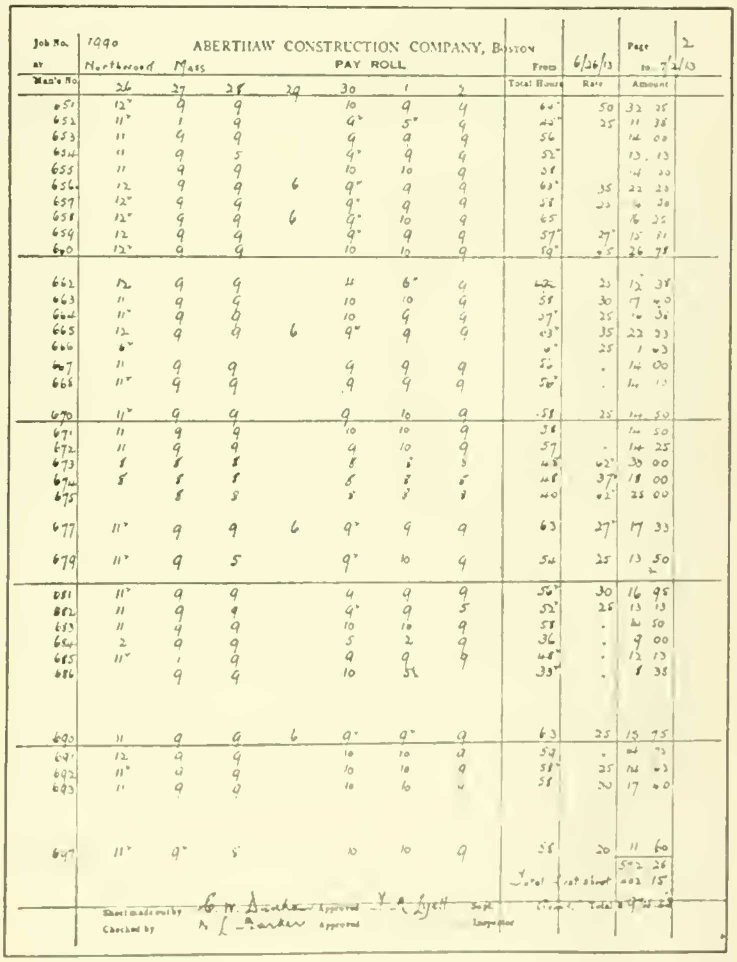
at Northwood Mass PAY ROLL From 6/26/1913 to 7/2/1913

Man's No.	26	27	28	29	30	1	2	Total Hours	Rate	Amount	
Lyell	9	9	9		9	9	9	54	37 ..	37 ..	
Barker	11	9	9		10	9½	9	57½	18 ..	18 ..	
Dunham	11	9	9		10	10	9	58	13 ..	13 ..	
[illegible]	9	9	9		10	10	9	56	36 ..	36 ..	
[illegible]	9	9	9		9	10	9	55	25 ..	25 ..	
611	9	9	9		10	10	9	56	30	16 80	
614	9½	9	9		10	10	9	56½	50	28 25	
615	5							5	30	1 50	
616	9	9	9		8	10	9	54	45	24 30	
617	9	9	9		10	10	9	56	50	28 00	
618	9	9	9		10	10	9	56	.	28 00	
619	9½	9	9		10	11½	9	58	45	26 10	
620	9	9	9		9½	11½	9	57	.	25 65	
621	9	9	9		10	10	9	56	.	25 20	
622	9	9	9		10	10	9	56	.	25 20	
623	10	9	9		10	10	9	57	25	14 25	
639	11	9	5		10	9	9	53	30	15 90	
640	11½	9	9½		10	10	9	60	25	15 00	

Sheet made out by G. H. Dunham Approved [illegible] Lyell Supt. Total $

Checked by H. L. Barker Approved Inspector

Fig. 8. The Pay Roll (Continued in Fig. 9).



Job No. 1990 ABERTHAW CONSTRUCTION COMPANY, Boston Page 2

at Northwood Mass PAY ROLL From 6/26/13 to 7/2/13

Man's No.	26	27	28	29	30	1	2	Total Hours	Rate	Amount	
651	12½	9	9		10	9	9	64½	50	32 25	
652	11½	1	9		4½	5½	9	[illegible]	25	11 38	
653	11	9	9		9	9	9	56		14 00	
654	11	9	5		9½	9	9	52½		13 13	
655	11	9	9		10	10	9	58		14 50	
656	12	9	9	6	9½	9	9	63½	35	22 23	
657	12½	9	9		9½	9	9	58	[illegible]	[illegible]	
658	12½	9	9	6	9½	10	9	65		16 25	
659	12	9	9		9½	9	9	57½	27½	15 81	
660	12½	9	9		10	10	9	59½	45	26 78	
662	12	9	9		11	6½	9	[illegible]	25	12 38	
663	11	9	9		10	10	9	58	30	17 40	
664	11½	9	9		10	9	9	57½	25	[illegible] 38	
665	12	9	9	6	9½	9	9	63½	35	22 23	
666	6½							6½	25	1 63	
667	11	9	9		9	9	9	56	.	14 00	
668	11½	9	9		9	9	9	56½	.	14 13	
670	11½	9	9		9	10	9	58	25	14 50	
671	11	9	9		10	10	9	58		14 50	
672	11	9	9		9	10	9	57	.	14 25	
673	8	8	8		8	8	8	48	62½	30 00	
674	8	8	8		8	8	8	48	37½	18 00	
675		8	8		8	8	8	40	62½	25 00	
677	11½	9	9	6	9½	9	9	63	27½	17 33	
679	11½	9	5		9½	10	9	54	25	13 50	
681	11½	9	9		9	9	9	56½	30	16 95	
682	11	9	9		9½	9	5	52½	25	13 13	
683	11	9	9		10	10	9	58	.	14 50	
684	2	9	9		5	2	9	36	.	9 00	
685	11½	1	9		9	9	9	48½	.	12 13	
686		9	9		10	5½		33½	.	8 38	
690	11	9	9	6	9½	9½	9	63	25	15 75	
691	12	9	9		10	10	9	59	.	14 75	
692	11½	9	9		10	10	9	58½	25	14 63	
693	11	9	9		10	10	9	58	30	17 40	
697	11½	9½	5		10	10	9	55	20	11 00	
										542 26	
								Total first sheet		402 15	
								Grand Total		$[illegible]	

Sheet made out by G. H. Dunham Approved [illegible] Lyell Supt.

Checked by H. L. Barker Approved Inspector

Fig. 9. The Pay Roll (Continued from Fig. 8).

by he could calculate from his original estimate the quantity of work done, and using this as a check. The practice on our work has improved a good deal since then and it is very seldom necessary now to make this check, but it is one that should not be neglected by one starting on a cost accounting system, as without this check the cost accounts may be rendered worthless by quantities either in excess or less than the actual amounts being reported. On one job, some years back, we found that a superintendent had reported 20,000 sq. ft. more forms than he had put up, with a view to making his costs look low and getting credit for economical work. Our estimates show what the total quantities should be, and an occasional survey and a final comparison will prevent any such errors being made now.

In concrete work the only item of labor that can be checked from the bills is the steel reinforcement, and every other quantity has to be obtained by scaling and computation. This is also true in most other branches of construction work, with the exception of structural steel.

(E) THE INQUIRY FORM.

We also use a brief standard form in case of any apparent mistakes on any of the reports received from the field. Often a man's rate is entered wrongly, or a different number of hours appears on the payroll to that on the time sheet, or time is reported with no quantities or quantities with no time. It is only by picking up all these mistakes at once and having them corrected that a system is kept going properly, and the men in the field, knowing that their work is carefully watched, are more keen in getting work done accurately. This may seem to be a trivial detail, but the writer's experience is that it is not possible to get accurate and careful work from men in the field unless careful attention is paid to the smallest items, and any inaccuracies or omissions are promptly followed up. If the men on the job are made to realize that their works is important and really counts for something they will be a good deal more careful and eager to co-operate with the office in matters like this.

Roofing Materials for Buildings.

The type of roofing adopted for buildings is a very important item, as a poorly designed or poorly constructed roof will result in large maintenance and reconstruction costs. The following data on roofing materials were taken from the report of the Committee on Buildings of the American Railway Engineering Association.

In selecting a roofing there should be considered:

(1) Chance of leaks, due to character of construction.

(2) Probable life, including chance of damage by the elements and by wear from other causes

(3) Fire-resisting value.

(4) Cost of maintenance.

(5) First cost.

The important materials may be classified as follows:

(a) Bituminous substances, applied with felts made of rags, asbestos or jute

(b) Clay and cement products and slate.

(c) Metals.

As to manner of laying the roofing materials they may be placed in one of two classes: Materials for a flat roof, cemented together, as a coal-tar pitch-and-gravel roof or an ordinary tin roof; and materials for a steep roof, laid shingle fashion.

BITUMINOUS MATERIALS.

The common bituminous materials are: Coal tar pitch (the heavier distillates of bituminous coal); various asphalts (bitumens found naturally in the solid state); various petroleum products; and various animal and vegetable residues. Their peculiar value lies in the fact that they are practically insoluble in water, and that they are elastic, adhesive, and comparatively stable.

Coal tar pitch is easily affected by heat and cold, is not acted upon at all by water, is easily worked, and, if properly protected, is very stable. It should ordinarily be used as it comes from the still "straight run," of a consistency suitable to the climate and to proper application. *Water-gas tar pitch,* a by-product in the manufacture of water gas, which is enriched by gas from petroleum oils, resembles coal tar. It is inferior to coal tar pitch for roofing purposes, and materials made from it should only be accepted in the low-priced products. It has more value as a saturant of felts than as a coating.

The *asphalts* are unsuitable for use in their natural state. They are ordinarily fluxed with products of petroleum.

The *petroleums* found in this country vary considerably, and grade roughly in quality, according to location from East to West. The California oils, with their asphaltic base, furnish materials especially valuable for roofing. The blowing of air through a heated still of certain petroleum products produces "blown oils," which, while somewhat lacking in adhesive properties, are not easily susceptible to atmospheric changes and are valuable especially for roofing coatings.

A single asphalt fluxed with a single oil is for most purposes a crude and unsatisfactory material. To secure the best results for any desired purpose, several oil and asphaltic substances must ordinarily be compounded. This requires skill and experience. Those properly made are for certain conditions invaluable, particularly for ready roofing, for which tar products are not suited.

The asphalt and petroleum products are not so readily affected by heat and cold as is coal-tar pitch, and lesser amounts of them are necessary to get good results. They are more expensive, require more skill in handling, and, when protected, some at least are to some extent liable to lose their life by drying out of the oil fluxes. Unprotected, they do much better than does coal tar.

FELTS.

The bituminous substances are used with felts whose qualities considerably affect the roofing. The ordinary felt is made of rags, mainly cotton. "Wool Felt" is a misnomer. Asbestos felts, as compared with the rag felt, act less as a carrying medium for the bitumens, but rather as a protection to the layers of bitumen. They are not suited for use with coal-tar pitch, but are not injured by hot asphalt. They are more expensive than rag felts, but have some peculiar and valuable qualities. Burlap made from jute decays easily when not protected. It is used in a few ready roofings, with rag felts, to increase their tensile strength, the need of which is not generally agreed to.

BUILT-UP ROOFS.

The bituminous roofings come ready to lay, or can be built up on the roof, using layers of saturated felt mopped with pitch, and properly protected.

The built-up roof is especially valuable for flat surfaces. It can be made as heavy as desired and if properly laid and of good materials it gives a roofing which by long experience has been shown to be economical and efficient. Where the roof is to be subjected to wear and where the character of the construction warrants the expense, flat tiles or bricks should be used as a protective coating to the roofing instead of gravel or slag.

For the flat roof built under average conditions, coal-tar pitch is recommended in preference to asphalt products. It is more easily handled, requiring less skill, and while more material is necessary it is still cheaper, and in our opinion more certain results can usually be expected from its use when laid by the average contractor. The large amount of material, although heavy, has an insulating value. Good results, however, can be expected from built up roofs using good asphalt compounds where laid by skilled workmen.

When the slope of the roof is over 3 ins. to the foot, the application of a built-up roof becomes more difficult for both coal-tar and asphalt, it being harder to get even mopping and there is more chance of accident for the men. The desirable straight run coal-tar pitch cannot be used, it being necessary to add some stiffening material which is supposed to affect somewhat the life of the pitch. This must not be done except under supervision skilled in such work, and especial care must also be taken in the selection and application of the stone or slag coating.

Built-up roofs with a ready roofing for the coating sheet are proposed by various manufacturers. They should have their best value for steep slopes.

The advantages of a coal-tar pitch built-up roofing are such that it is recommended where a permanent roof is desired and where the character of the structure allows the building to be so designed as to allow its use. A flat roof makes an economical structure and has small fire hazard. A pitch of from ½-in. to 1-in. to the foot is better than anything steeper. With proper materials and application a life of from 15 to 20 years can be expected with a flat roof.

No contracts should be made for a built-up roof without a complete and positive specification which includes flashings, and the contract prices should not be less than those of the materials specified plus a reasonable amount to cover the cost of laying and profit. Thorough inspection of workmanship and material is recommended.

READY ROOFING.

The ready roofing has better value for the steeper roofs than for those of small pitch. It averages much cheaper than the built-up types. Most kinds to get a fair life require occasional recoating. For flat slopes they are hard to lay absolutely tight, and they are not economical for a permanent structure, but on slopes of from 3 ins. to the foot up their use is more justifiable.

Ready or prepared roofings are recommended for use on small, temporary and other buildings, where the cost, considering maintenance, of more expensive roofings is not justified. They are also of value for steep slopes where a built-up coal tar cannot be used, and for locations where the skilled labor necessary for a built-up roof is not available. The steeper the slope the greater their relative value and the wider their economical field. The heavier varieties are in general the more desirable because of their chance for longer life and their greater fire-resisting value. In making selections the reliability of the manufacturer, service tests and the cost should be governing factors.

On the steeper slopes the use of ready roofing shingles properly reinforced so as to prevent curling up at the corner and fraying on the exposed edges and laid shingle-fashion is growing. They are supposed to give better results than the rolled goods, but cost more. They would seem at least to be worthy of investigation.

SLATE AND TILE.

Slate makes a good roof if of good quality and properly watched. It breaks easily and cannot be walked on without danger to the slate.

Tile of good quality gives good results. It is not so tight as slate, but does not break easily. It has architectural value, and its use is growing, with improvement in the product and in the variety of colors.

Slate and tile of suitable quality, properly protected and fastened, can be recommended on roofs with a pitch of 6 ins. to the foot or over, where expense is not the governing feature and where they aid in producing the desired architectural effect, except that where there is much chance of driving snow 8 ins. to the foot should be the flattest slope allowed.

ASBESTOS SHINGLES.

Shingles of asbestos and Portland cement are of value. They have some elasticity and can be driven down tight.

WOOD SHINGLES.

Wood shingles are not now desirable for a railroad structure.

CEMENT TILE.

Small cement tile are not considered of much value, being brittle. Large cement tiles einforced, laid without sheathing directly on he purlins are in use on shops and freight ouses and seem to have considerable merit. Glass can be introduced into them, thus avoiding the expense of skylights. We are not ready to recommend them for plastered or heated buildings or offices where an occasional slight leak would be disastrous.

METAL ROOFINGS.

Metallic roofings with steel as a base are not recommended for general use on permanent buildings. They require continual maintenance.

Galvanizing of steel seems to be well worth the expense. Tests of lead covered steel sheets indicate good results. Large sheets of corrugated galvanized steel can sometimes be used economically where the building is not to be heated.

Small metallic shingles of either copper, tin, galvanized steel plate or specially pure iron are not recommended for general use. They are very light in weight and serve a purpose, particularly in the dry climate of the Southwest.

In using metals, every effort should be made to secure those of good quality. The pure irons have value. Their virtues have perhaps been overstated, but they are not expensive, and experience seems to indicate considerable economy by their use as a substitute for wrought-iron and steel.

Copper, lead, zinc and Monel metal are used for roofing, but they are not of value for ordinary railroad structures.

GENERAL COMMENTS.

In the laying of all roofings thoroughness in the preparation of flashings and work around openings is of vital importance.

To get a satisfactory roof there must be a stable structure, careful attention must be given to the design of gutters, and with some types particularly there must be systematic inspection and regular repairs. In buying a roofing its fire resisting qualities, to a considerable extent depending on the quantity of material as well as its quality, are of great importance. A building covered with a heavy coal-tar pitch-and-gravel roofing is a better fire risk than one covered with corrugated steel sheets, or with a light ready roofing. The practice of depending merely upon guarantees in selecting roofings cannot be trusted to secure proper results. It does not pay to put a cheap roof on a good building. The annoyance and indirect expense occasioned by leaky and short-lived roofs are rarely compensated for by any possible saving in first cost.

Water Works

New Facilities for Storing, Conveying and Weighing Lime Used at Columbus, O., Water Purification Works.

At the time the Columbus, Ohio, water purification plant was put in operation all of the chemicals for use in softening and purifying the water were handled and stored in canvas bags. The original chemical storage house consists of one large room 59 ft. 8 ins. wide by 105 ft. long and has an estimated capacity of 900 tons. The first consignment of lime, consisting of several hundred tons were delivered in bags, each containing a net weight of 100 lbs. The bagged material was stored in the chemical storehouse, but before it could be used it had air slaked and almost all of the bags had burst, demonstrating conclusively the inadvisability of storing lime in bags. Further trouble was also experienced, while using lime delivered in bags, in maintaining a uniform application of the chemical to the water and it was found that the bags of lime varied in weight from 10 to 15 lbs. per bag.

After this experience of storing sacked lime, it was decided to buy lime delivered in box cars loaded in bulk. A reserve stock of lime amounting to about 60 tons was stored in steel drums provided with an 8-in. screw top. Arrangements were also made with the lime contractor whereby he was to ship the lime at a uniform rate of not less than 40 nor more than 125 tons per week as directed by the chemist in charge of the water purification works. As soon as it was received, the lime was sacked, 90 lbs. to the sack, by the workmen at the purification works, and it was planned to regulate the lime supply and shipments so that the lime could be used as soon as received.

The amount of lime necessary to soften the water changes from day to day, and during flood times or after heavy rains, the hardness of the river water drops from 300 to 400 p.p.m. to 100 to 150 p.p.m. These changes sometimes occur in less than four hours' time. Consequently there were often six or seven cars of lime on the siding at the plant or in transit in excess of that which could be consumed. Some of these cars were held as long as two weeks before being unloaded because there was no available space to store the excess material.

This condition of affairs resulted in a loss to the city, first because of the demurrage on cars and second because the lime air slaked and lost strength. The cost of sacking and weighing the lime was high. It required five men working nine hours to sack and weigh a car of lime containing 20 tons; and after the lime was sacked it took from 1½ to 2 hours to elevate the lime to the third floor of the head house, where it was again lifted by hand to the lime shakers. The work of sacking lime from cars also proved to be a very disagreeable job, especially in hot weather, on account of the lime dust slaking on the sweaty hands and the faces of the men. The slaking of the lime dust often resulted in severe burns.

The storage capacity for lime having become inadequate and the method of handling unsatisfactory, Mr. Charles P. Hoover, chemist in charge of the purification works, and Mr. C. J. Clarke, engineer in the city water works department, prepared plans and specifications for new equipment for handling lime. A brief description of the plant, as finally adopted, is here given from information contained in the report of Mr. Hoover for 1913.

NEW EQUIPMENT.

The equipment consists essentially of a concrete bin, elevated on columns over the railroad tracks located between the head house of the water purification works and sedimentation basins; a V-bucket conveyor and elevator for elevating the lime to the level of the head-house roof; a screw conveyor for conveying it across and into the head house, where it is dropped into tanks; and three scales for automatically weighing and dumping the lime into the old slaking tanks below.

Storage Bin.—The storage bin consists of a concrete hopper 25½ ft. square with bottom slopes of approximately 38° with the horizontal, heavily reinforced to take the direct pull due to the weight of the lime and the horizontal thrust due to the bulging effect. The main weight is carried on four reinforced concrete beams, supported in turn by eight columns with flared footings, which take bearings on the solid gravel under the track bed. The center column takes part of the load of the hopper and is cored inside with an 18-in. square opening used as a chute to reclaim lime from the storage bin and deliver it by means of the elevator and conveyor to the tanks in the head house.

The lime may be taken directly from the cars to the tanks without passing into the storage bin. The storage bin has a capacity of 220 tons of lime, or about 22 days' supply, which gives ample storage for any emergency.

Conveying Equipment.—The conveying equipment consists of a continuous V-bucket conveyor and elevator with a capacity of 50 tons per hour, running in a concrete trough between the tracks and through chutes each side of the bin, so that it is enclosed throughout its entire length. The lime is discharged from this elevator into a 12-in. screw conveyor which extends into the roof of the head house and discharges into the top of three tanks which are supported on the third floor of the head house on angle columns and extend through the ceiling into the roof space above.

The V-bucket conveyor and the screw conveyor are operated by an 18-HP. Westinghouse direct current, 220-volt motor, situated in the roof of the storage bin. The tanks are constructed of ¼-in. steel plate, are 8 ft. square, with hopper bottoms, and have a combined capacity of approximately 22 tons, or about two days' supply.

Lump lime, averaging about 2 ins. in size, is fed from the tanks into automatic weighing scales, which automatically weighs the lime into the slaking tanks. The slaked lime discharges from the slaking tanks into the lime solution tanks below. Milk lime, of required strength, is made by the addition of water. The milk of lime is kept well stirred by means of paddles and jets of compressed air and is measured into the water by a chemical feed regulator. Later it is planned to pipe the slaked lime directly from the slaking tanks into the water, thus by-passing the lime solution tanks and the chemical feed regulator.

The writer is indebted to Mr. W. N. Goodman for the following description of the automatic scales.

Automatic Scales.—The three automatic scales are alike. Each is built on the equal balanced-beam principle. On knife edges at one end of the scale beam is suspended the weight hopper and at the other end of the beam, also on knife edges, is suspended the weight box or pan, in which are placed U. S. standard weights to the amount of 75 lbs. when that quantity is desired to be weighed. If a 100 or a 150-lb. weight is required, then the actual weight desired to be discharged is placed in the weight box, or the excess of weight over 75 lbs. can be added by a sliding counter poise working on the graduated lever.

The lime is fed to the weigh hopper of the scale down a sloping chute in which is placed a reciprocating plate feeder to keep the lime from blocking. This feeder is in action only while the scale is filling. It is motor driven by an electric motor and as soon as the scale has received its load the motor automatically stops.

After being fed down the inlet chute the lime passes through a gate of the undercut type. When the weigh hopper has received four-fifths of its load the gate partially closes, the last one-fifth being dribbled into the hopper. When the exact amount has been received into the hopper, the gate closes and is locked by a toggle mechanism. The scale hopper holds the load until it is time to discharge it.

Columbia School of Mines to Celebrate Fiftieth Anniversary.—The fiftieth anniversary of the Columbia School of Mines is to be celebrated May 28, 29 and 30, and it is expected that a notable gathering of engineers will result. A committee from the trustees, faculty and alumni has been at work on the plans for some time and an interesting program has already been formulated. Dean Frederick Arthur Goetz of the School of Mines, Engineering and Chemistry will make a trip to all the large cities of the United States with a view to interesting alumni and inviting them to aid in the celebration.

Locating Leaks in Water Mains by Means of the Water Hammer Diagram.

When the valve at the end of a long pipe line is closed suddenly, great pressures may be caused. The term water hammer has been applied to this phenomenon. If the valve could be closed instantly all of the water in the pipe would not be stopped at the same instant. The layer nearest the valve would stop first, then the next layer and so on until the impulse had traveled through the entire pipe line. As each layer of water is brought to rest its pressure will of course be increased. The velocity of the transmission of the pressure wave will be the same as the velocity of transmission of sound in the water in the pipe, and will vary between 3,400 and 4,700 ft per second, depending upon the material of the pipe and upon the ratio of the thickness to the diameter of the pipe.

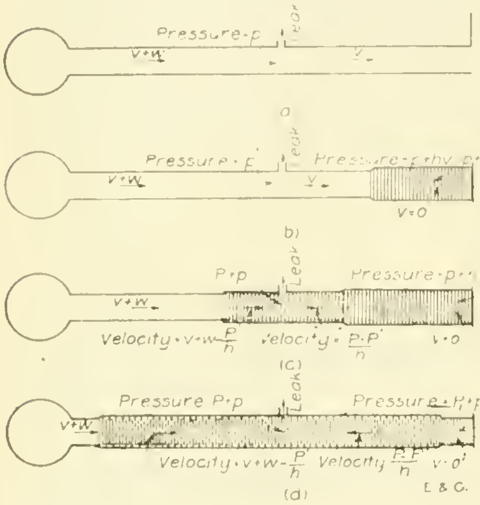


Fig. 1. Diagrams Showing Condition at Different Times in Pipe Line Subject to Water Hammer and Containing a Leak.

(a) Pipe line containing leak.
(b) Conditions a short time after valve at end of line is closed.
(c) Conditions a short time after pressure wave has passed leak.
(d) Conditions a short time after wave of reduced pressure has reached valve.

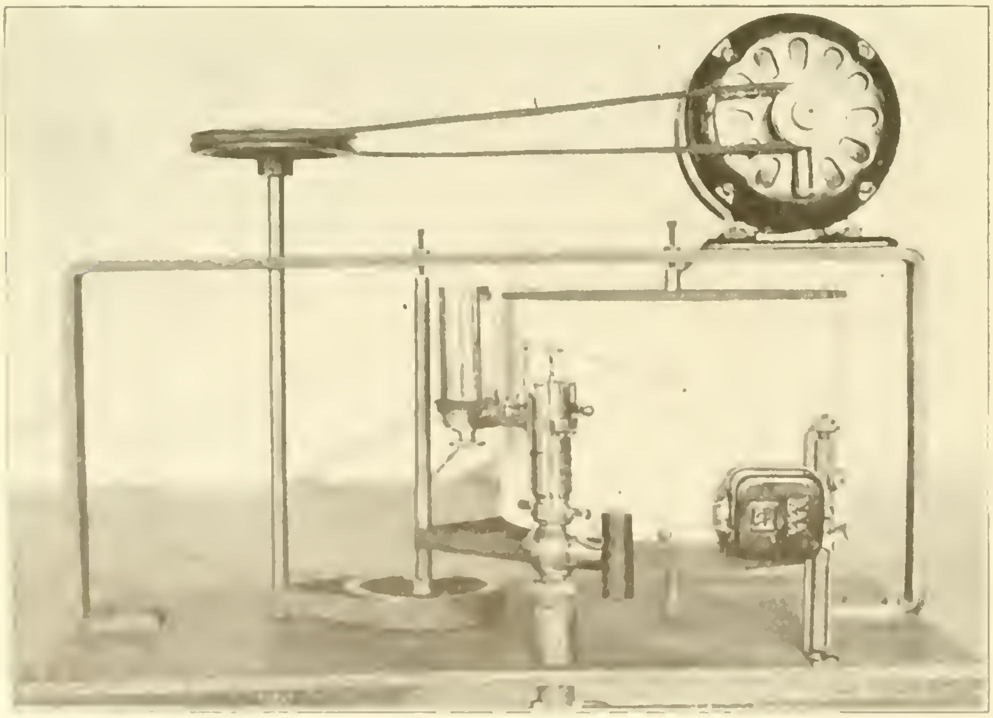


Fig. 2. View of Apparatus for Obtaining Water Hammer Diagram.

It has been found that for any given pipe, the amount of the water hammer pressure is a constant times the extinguished velocity. The value of this constant (also called the water hammer coefficient) varies directly with the velocity of transmission of the pressure wave, and for cast iron pipe used for water supplies has values between 45 and 63. For cast iron pipe between 6 and 16 ins. in diameter, the average value of the constant is about 55. That is, the water hammer pressure caused by the sudden closure of a valve at the end of a long pipe line, in pounds per square inch, is 55 times the velocity of the water in the pipe before the valve was closed, in feet per second. The use of the water hammer diagram for locating leaks in water mains was described by M. L. Enger, assistant professor of theoretical and applied mechanics, University of Illinois, in a paper before the Illinois Water Supply Association, which we reprint as follows:

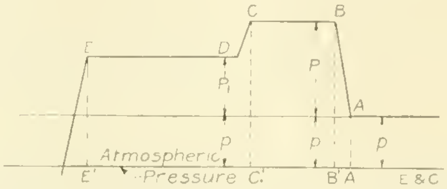


Fig. 3. Characteristic Features of Water Hammer Diagram Taken at End of Pipe Line Containing a Leak.

Figure 1a represents a pipe line in which there is a leak. The flow between the source (reservoir or large pipe) and the leak is $v - w$ feet per second, and between the leak and the valve is v feet per second. Figure 1b shows the conditions in the pipe line a short time after the valve at the end is suddenly closed. The velocity of the water near the valve has been extinguished and its pressure increased hv pounds per square inch (h being the water hammer coefficient). If the distance from the valve to the leak is l feet and the velocity of the propagation of the pressure wave is Z feet per second the pressure wave will reach the leak l/Z seconds after the valve closed. Since the original pressure at the leak allowed a quantity of water equal to Aw cubic feet per second to escape, it is evident that a higher pressure will cause a great quantity to flow. The extinguished velocity between the leak and the source will therefore be less than v feet per second. Hence the water hammer pressure generated in this part of the pipe line will be less than hv pounds per square inch. A wave of reduced pressure will therefore travel from the leak toward the valve. Figure 1c shows the conditions a short time after the pressure wave has passed the leak. The wave of reduced pressure will reach the valve $2l/Z$ seconds after the valve closed. Figure 1d shows the conditions a short time after the wave of reduced pressure has reached the valve.

The water hammer diagram is a graphical representation of the pressure in the pipe line near the valve for a time after the valve is closed. In the experiments made by the writer the diagram is obtained by having the pencil of an indicator trace on a sheet of paper wrapped around a drum driven at a uniform rate by an electric motor. Another pencil attached to an electro-magnet makes a time record. A photograph of the apparatus is shown in Figure 2.

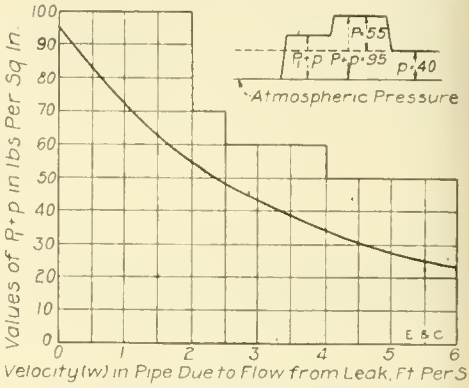


Fig. 4.—Diagram Showing Velocity in Pipe Due to Flow from Leak.

Figure 3 shows the characteristic features of a water hammer diagram taken at the end of a pipe line in which there is a leak. The first rise of pressure as the valve begins to close is shown at A. The indicator pencil reaches B when the valve is fully closed. The pressure then remains practically constant until the effect of the leak is registered at C. The distance A'C' represents the time required for the pressure wave to travel from the valve to the leak and back to the valve. If the velocity of transmission of the pressure wave is known, the distance from the valve to the leak is easily computed. The difficulty in the use of this method is in the determination of the velocity of transmission (Z) of the pressure wave. The velocity of the pressure wave will vary somewhat according to the amount of air in the water. Another method which avoids the necessity of determining the value of Z is as follows: When the indicator pencil reaches E the first relief of pressure due to the source is felt. The distance A'E' therefore represents the time required for the pressure wave to travel from the valve to the source and back to the valve. If the length of the pipe line from the valve to the source is L, the distance from the valve to the leak can be determined by proportion.

$$l : L :: A'C' : A'E'$$

In the author's experiments much more consistent results were obtained by this method than by the use of the velocity of transmission of the pressure wave and the time required for the pressure wave to go from the valve to the leak and return as scaled from the diagram.

The quantity of water discharged from the leak can also be determined from the water hammer diagram. The expression for the velocity of flow in the pipe due to the leak is:

$$w = \frac{\frac{P - P_1}{h}}{\left(\frac{P + P_1}{2p} - 1\right)^{0.5} - 1}$$

in which P is the amount that the pressure is increased due to water hammer, P_1 is the amount that pressure at the valve is above the original pressure after the return wave from the leak reaches the valve; P is the original pressure at the valve; h is the water hammer coefficient..

The following values are taken from the experiments made by the writer in 1906. The last two were measured from the second diagram shown in Figure 5.

Calculated distance, ft.	Actual distance, ft.
64	72
70	72
371	381
385	381
375	381
262	265
265	265
116	113

A number of values have been computed from the equation, and the results have been plotted in Figure 4. It was assumed that $P = 55$ lbs. per square inch, $p = 40$ lbs. per square inch, and that $h = 55$. It will be seen from an examination of the curve that even small leaks will produce a noticeable fall of pressure on the water hammer diagram.

The first suggestion that the water hammer diagram could be used to determine the location of a leak was made by Professor Joukovsky as the result of a series of experiments made in 1897 and 1898 for the water works department of Moscow, Russia. He published a monograph (Stoss in Wasserleitungsrohren) in 1900. A translation of this paper, somewhat modified, was published in the Proceedings of the American Water Works Association in 1904. Experiments were made by the writer in 1906 in the hydraulic laboratory of the University of Illinois. Figure 5 shows two diagrams taken at that time.

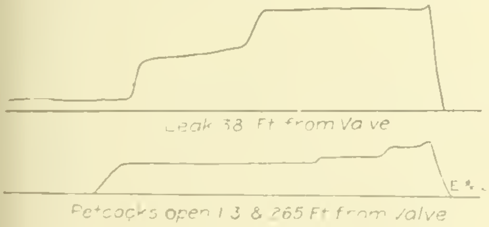


Fig. 5. Water Hammer Diagrams Taken for Pipe Lines Containing Leaks.

SUGGESTIONS FOR USE OF METHOD.

In using this method for determining the location of a leak, the following suggestions are made. The quick closing valve should be at the end of the section of pipe to be tested. This can be accomplished by tapping the main close to a valve, the valve in the main being kept closed during the expirement. The pipe leading from the main to the quick closing valve must be large enough that a water hammer pressure at least as great as the static pressure can be caused by the sudden closure of the valve. The indicator should also be connected at this point of the main, or to the pipe containing the quick closing valve. If possible the method of proportional distances should be used. The distance to the source (large main) should be measured. A hydrant partly open will make a good reference point in case the main on which the experiments are being made is very long. When the method of proportional distances is used it is not necessary to know the speed of the paper. It is only necessary that the paper travel at a uniform speed while the diagram is being taken.

Data and Discussion on Water Softening for Railroads.

Scale formation in large locomotives, of 2,500 sq. ft. of heating surface and evaporating over 5,000 gals. of water per hour, greatly shortens the life of the boiler and appurtenances, necessitates frequent and costly repairs with resulting considerable loss of time for expensive machinery, and the leaks caused by the opening of flue joints, due to scale formation, causes delays to traffic. The scientific investigation of this economy is of comparatively recent development, that is, it has been worked out most effectively in the past 15 years. It is no longer an experiment although as with many innovations, it is still regarded in a few places with suspicion. While the process originated in England, if the report of the International Railway Congress of 1910, at Berne can be taken as a criterion, the United States now leads the world in water softening efficiency. A discussion of present practice in water softening by American railroads was presented, with data, by Mr. R. C. Bardwell, chemist of the Missouri Pacific Railway, in his recent paper before the Illinois Water Supply Association. The paper, slightly condensed, follows:

On the generally softer waters in the eastern part of the country the process was first tried with most gratifying results. On the harder and heavy scale waters of the middle west and western districts it is now regarded as a practical necessity. In cases where it has been successfully installed and competently operated the increase in the life of flues has amounted to from 75 per cent to 150 per cent with corresponding benefits to the machinery and reduction of losses.

The report of the committee on water service for the 1914 convention of the American Railway Engineering Association shows that on six representative trunk lines through the middle west there are now in service 237 complete treatment plants and 111 soda ash partial treatment stations, ranging from 5 complete plants on one road to a maximum of 112 on another, and from no soda ash plants on several to 86 on one. Besides this several railroads arrange for the placing of soda ash directly into the engine tanks at the terminals. This is conducive to foaming conditions, but the economy derived from the scale removal more than justifies the foaming complaints.

On the Missouri Pacific there are at present 45 complete water softening plants in operation, the majority being on the hard waters west of the Missouri River. The average amount of water treated per year, reducing the hardness so that it will form practically no scale, is 1,692,000,000 gals. The total average amount of scale removed from this water is 5,537,000 lbs., which would make over 110 carloads at 50,000 lbs. each: a considerable amount when it is remembered that but for treatment this scale would have to go through the engine boilers and most of it would have to be removed by hand. The total annual cost for the above treatment, interest and depreciation on plants is about $65,000. Conservative figures show, however, that with this expenditure there is a net saving of about $105,000 from cutting down the following losses alone:

1. Frequent renewal of flues and other parts of boiler account of scale accumulation and injury to flue ends from repeated caulking. 2. Labor caulking flues and other engine house boiler repairs. 3. Loss of engine time during boiler and firebox repairs. 4 Loss of fuel due to insulating effect of the scale on the flues and other heating surfaces.

Besides the foregoing there are the indeterminate benefits in the road performance of locomotives by reducing failures and interruptions to traffic with the reduction of the number of locomotives required for a given traffic.

At one terminal approximately 18,000,000 gals of water are treated monthly, eliminating 5 lbs. of scale per 1,000 gals. for a total of about 80,000 lbs. per month. The cost of chemicals for this is about 4 cts. per 1,000 gals. or a total monthly bill of about $720. The length of life of flues using straight raw water before the installation of the softening plant was from eight to twelve months with serious trouble on account of frequent leaks. The locomotives now using the straight treated water average 18 months between shopping, and the trouble with leaky flues is practically eliminated.

At another terminal shop about 30,000,000 gals. of water are treated monthly, removing 3 lbs. of scale per 1,000 gals. for a total of 90,000 lbs. Prior to the installation of the softener, five boilermaker days and four boilermaker nights were required caulking flues. After the treating plant was put in operation this was reduced to one-man days and one-man nights. The saving in this one item alone is reported to be $5,712 per year on an $8,000 investment. The life of flues was increased from 8 to 15 months.

In most cases where softening plants are installed, the foaming conditions are increased, especially for the first few days after the treatment is started due to the fact that the old scale in boilers is loosened and, falling off, increases the amount of suspended matter making a dirty boiler which seems to be the most important of the contributory causes of foaming.

To soften a water to three grains per gallon, or less, of incrusting salts, which is the object in most cases, demands a purity in this respect of 3 parts in 58,311 or 99.995 per cent. This merely shows the deceptiveness of reporting water softening on the percentage basis, for although sounding complex it is a comparatively simple matter. However, each individual water has its peculiarities such as the temperature at different seasons and the difference in content of magnesium salts which slow up the reaction and without due allowance for same leave a milky water for a final product. Creek waters of course change with the precipitation and I have also found that in some parts of the country well waters become softer in winter while others get harder.

So far the development of water softening on a large scale has centered around lime and soda ash as being the chemicals which produce the results for the least cost. However, barium hydrate is the ideal reagent on account of its leaving no detrimental byproducts as in the case of soda ash. So far the expense has retarded its development. Numerous boiler compounds through the high degree of exploitation obtained have been tried by a few railroads but investigation would show that even where the work desired was done the cost greatly exceeds that of the recognized methods. Inert powdered graphite is now being largely exploited as a cure-all for scale troubles, but its economical merits are yet to be proven. The "sunlight on corrugated aluminum" patent, which was supposed to render the scale inert in the water without removing it chemically has been tried at several points in this section and seems to have proven unworthy. The Permutitt water softener, the artificial zeolite, will never be a universal success in railroad work account of the replacing of incrusting carbonates with sodium carbonate, that is, waters which are sufficiently hard to warrant the expense of treatment, as a rule contain sufficient incrusting carbonates, which if replaced by sodium carbonate could not be used on account of foaming properties. Therefore it would seem that common lime and soda ash will continue to remain in service.

Procedure in the Meter Division of the Detroit Water Works Department.

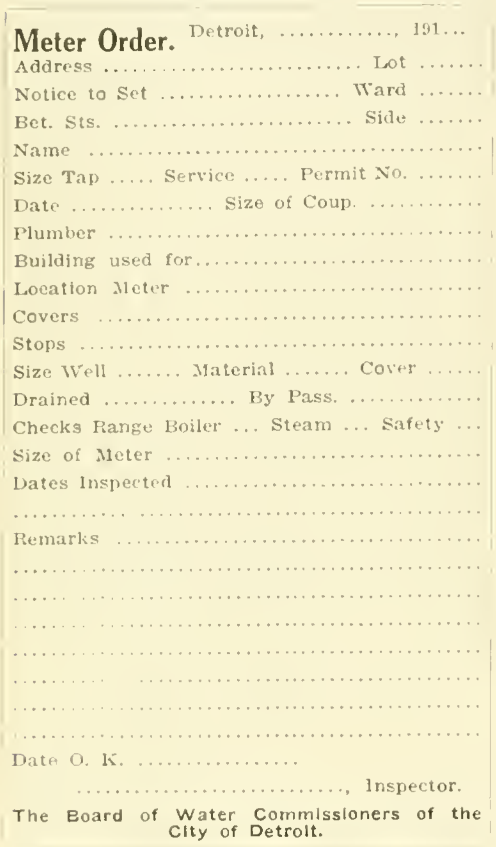
The present article describes procedure in the meter division of the Detroit water works department with special reference to the purchasing, testing, setting, reading and repairing of meters. The forms for keeping the records of the division are illustrated herewith. The article is based upon information furnished this journal by Mr. I. J. Clancy, chief clerk of the iron pipe department.

Shop and Equipment.—The meter shop is in the storage yard which is the distributing center for the supplies and the headquarters of the construction and maintenance divisions of the distribution system department. The shop, first occupied in January, 1911, is a two story brick building. It is electric lighted and steam heated. In this shop are housed the meter testing outfit and the repair benches and meter parts. The power equipment, described in this article, is also housed in this building.

The chief work at the shop consists of the testing and repairing of meters, but all the machine work for the construction and maintenance departments which can be handled by the size of the equipment in the shop is there performed. The department automobiles are also repaired in this shop.

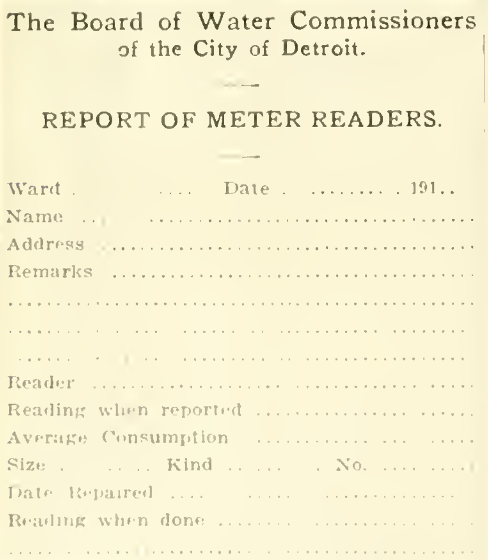
Card Records.—In the office, which occupies a portion of the space on the first floor of the shop building mentioned, a card system for the meter division is maintained. Six forms are illustrated herewith in Figs. 1 to 6. Figure 1 is the form used by the inspection department in reporting that arrangements have been made to receive a meter. Figure 2 shows the form used by the meter readers in reporting to the repair department that a certain meter has stopped or is faulty in registration. Figure 3 shows the card used in listing meters in stock and in service; Fig. 4 the card for recording test data; Fig. 5 the card showing details of meter location and setting, and Fig. 6 the card used in reporting to the meter reading department that meters have been set or removed, and for recording repairs to meters.

Purchase and Testing of Meters.—Meters are purchased on contracts awarded after receiving competitive bids. Each new meter is tested and if its registration is in error by as much as 3 per cent it is rejected or repaired at the maker's expense. Meters up to and including 2 ins. in size are tested on a concrete table with a capacity of 25 ⅝-in. or 10 2-in.



Meter Order. Detroit, ..., 191...
Address .... Lot ....
Notice to Set .... Ward ....
Bet. Sts. .... Side ....
Name ....
Size Tap ..... Service ..... Permit No. ....
Date .... Size of Coup. ....
Plumber ....
Building used for............................
Location Meter ....
Covers ....
Stops ....
Size Well .... Material .... Cover ......
Drained .... By Pass. ....
Checks Range Boiler ... Steam ... Safety ...
Size of Meter ....
Dates Inspected ....
....
Remarks ....
....
....
....
....
....
....
....
Date O. K. ....
............................, Inspector.
The Board of Water Commissioners of the City of Detroit.

Fig. 1. Form Employed in Detroit Water Department for Reporting Readiness at Premises to Receive Meter.

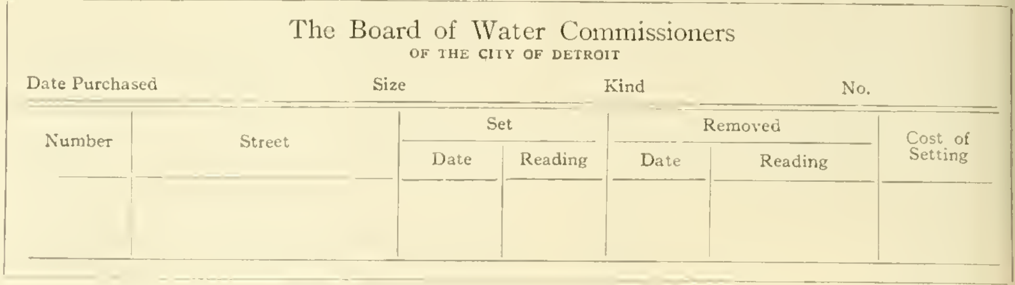


The Board of Water Commissioners
of the City of Detroit.

REPORT OF METER READERS.

Ward . .... Date . ... 191..
Name ....
Address ....
Remarks ....
....
....
....
Reader ....
Reading when reported ....
Average Consumption ....
Size . .... Kind .... . No. ....
Date Repaired ....
Reading when done ....
....

Fig. 2. Form Used by Meter Readers in Reporting to Repair Department the Stoppage or Faulty Registration of Meters, Detroit Meter Department.

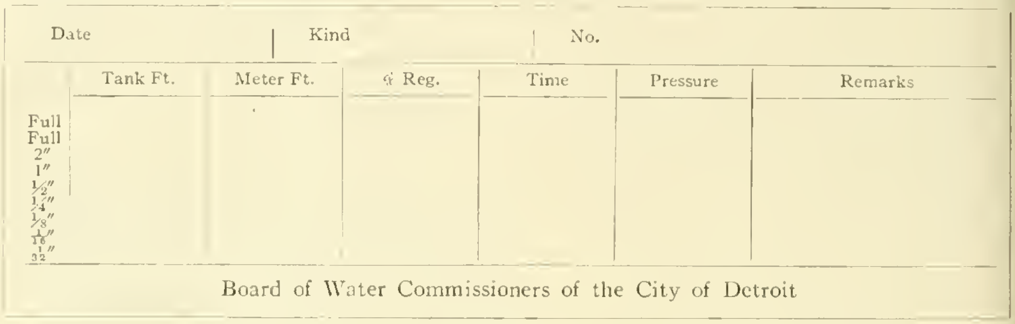


The Board of Water Commissioners
OF THE CITY OF DETROIT

Date Purchased Size Kind No.

Number	Street	Set		Removed		Cost of Setting
		Date	Reading	Date	Reading	

Fig. 3. Form of Card Used in Keeping List of Meters in Stock and in Service. These Cards Are Filed in Groups According to Make of Meters.

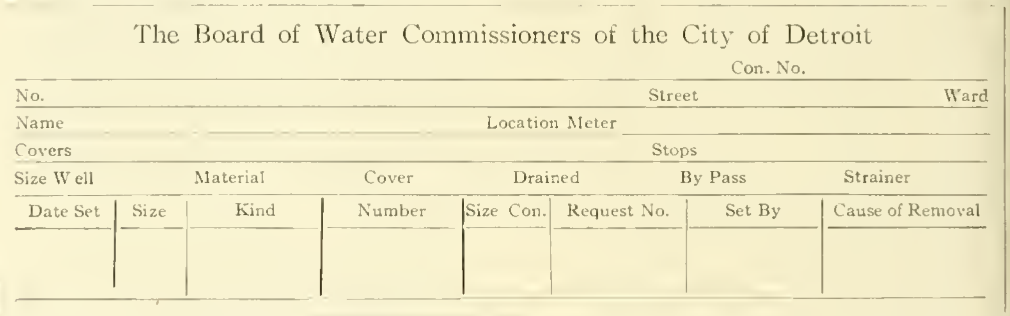


Date | Kind | No.

	Tank Ft.	Meter Ft.	% Reg.	Time	Pressure	Remarks
Full						
Full						
2"						
1"						
½"						
¼"						
⅛"						
1/16"						
1/32"						

Board of Water Commissioners of the City of Detroit

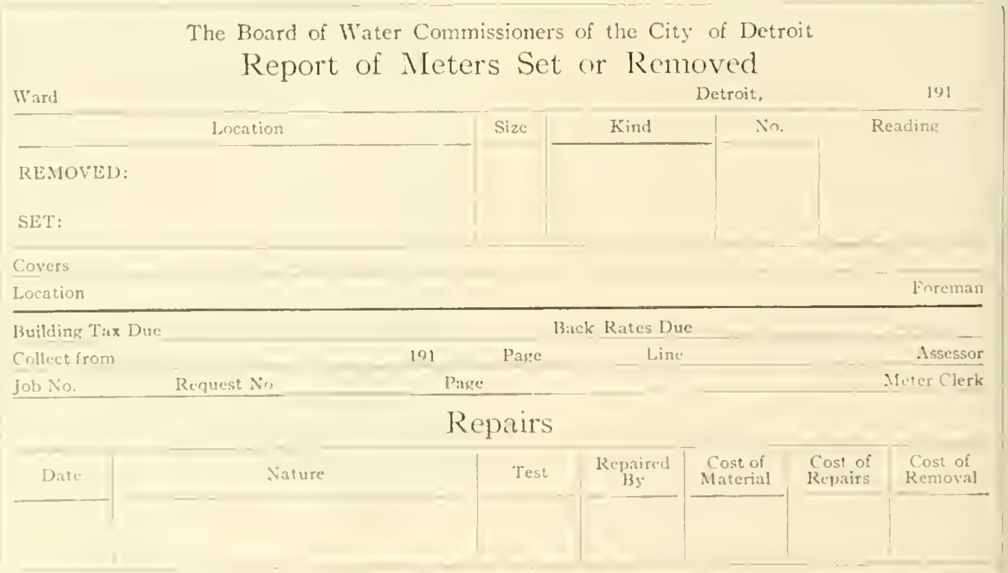
Fig. 4. Form of Card Used in Preserving Test Data for Each Meter.



The Board of Water Commissioners of the City of Detroit
Con. No.
No. Street Ward
Name Location Meter
Covers Stops
Size Well Material Cover Drained By Pass Strainer

Date Set	Size	Kind	Number	Size Con.	Request No.	Set By	Cause of Removal

Fig. 5. Form of Card Showing Meter Location. These Cards Are Filed According to Streets and Numbers.



The Board of Water Commissioners of the City of Detroit
Report of Meters Set or Removed
Ward Detroit, 191

Location	Size	Kind	No.	Reading
REMOVED:				
SET:				

Covers
Location Foreman
Building Tax Due Back Rates Due
Collect from 191 Page Line Assessor
Job No. Request No Page Meter Clerk

Repairs

Date	Nature	Test	Repaired By	Cost of Material	Cost of Repairs	Cost of Removal

Fig. 6. Card Printed on Both Sides. Upper Form Used in Notifying Reading Department of Setting or Removal of Meters. Lower Form for Recording Repairs to Meters, Detroit Water Department.

meters which are tested at the same time. Larger than these sizes are tested singly. Meters up to 2 ins. in size are given two tests of from 20 cu. ft. to 100 cu. ft. (according to the size of the meter) on full flow, and further tests of smaller flow down to 1/32 in. These flow reductions are made by inserting flanges with different sized orifices on the outlet side of the meter. Three-in. and larger meters are given two tests of 400 cu. ft. each on full flow. The flow is reduced in subsequent tests until the meter stops registering. During the tests of meters over 4 ins. in size they are not delivering their full capacity as we have only a 4-in. feed, but of course the accuracy is determined. After the water passes through the meters it empties into tanks of different capacities and when they are full the water is shut off and the meter reading taken.

In June, 1911, an endurance test for ⅝-in. meters was begun and 32 meters embracing 10 different makes were set up, 8 of which were placed vertically. The makes and numbers were carefully recorded and as fast as each one registered 100,000 cu. ft. it was taken off and tested for accuracy on flows from full to 1/32 in. and then returned to the endurance test stand. At the present writing eleven have stopped because of worn out internal gears and bearings and the remaining 21 have passed 1,000,000 cu. ft. The test will be continued until all the meters stop, when a full report of the findings will be compiled and preserved.

Power Equipment.—Both floors are equipped with traveling chain hoists of sufficient capacity to carry the largest meters from the door to the meter testing block or storage space upstairs. The first floor besides the office and testing outfit contains power equipment consisting of a 14-in. lathe, 24-in. self-feeding drill press, 14-in. sensitive drill press, 3-in. pipe cutter and threader, drill grinder, emery and buffing wheel and air compressor driven by a 10-HP. single phase motor.

The second floor is used for storing meters and parts, but as the installation of meters progresses, this storage space will give way to repair benches and the meters and parts will be stored in a stock room. This storage space is available because at present less than 13 per cent of the service connections are metered and the shop was built large enough easily to take care of the meter department if all connections were metered.

Meter Repairs.—Meters are repaired while on the connection when possible, but if it is necessary to remove and bring to shop, another meter is immediately put on, registering zero and is not replaced by the meter removed. This insures a metered connection at all times and saves a second trip. When stopped meters are removed for repairs, the reading department is notified and the period previous to the time of removal is estimated according to the consumption for the former quarter year. New meters are set after due inspection by our inspector and notification by the property owner of his readiness to receive same. Thus the rules of the board pertaining to location, etc., of meters are observed and no meters are set in unoccupied buildings. The item of sending men to set and repair meters is reduced to a minimum by the use of Ford runabouts equipped with bodies.

An Electrical Apparatus for Thawing Out Water Pipes, Successfully Used by Columbus, O., Water Works.

During the past two winters an electrical apparatus for thawing out frozen water services has been successfully employed by the Columbus, Ohio, water department. Since the device has demonstrated its practicability and value we are here reprinting the description of it as published in the annual report of the Division of Water for 1913.

The contemplated purchase of a 2½-ton truck for general use by the department suggested the mounting of a portable thawing equipment, inasmuch as it would only be used during the time of the year the truck would be least wanted for other purposes.

The facility for mounting and removing the equipment was unified as far as possible by bolting the various parts to a channel frame 8 ins. in depth.

The generator is of 30 kw. capacity, direct current, 100 volts maximum, 300 amperes, or 400 amperes at 75 volts; speed, 800 r.p.m. maximum.

The engine is of 50 brake horsepower at 800 revolutions per minute, four cylinder, four-cycle, direct connected to the generator by a flexible coupling. A special governor was designed which permits of governed control at all speeds between 250 and 850 r.p.m. Gasoline is used for fuel, supplied under 3 lbs. pressure from a 26-gal. tank.

A radiator sufficient in area to permit continuous operation of the engine at full load is mounted at the front end of the frame, and an air blast through the radiator is induced by means of a 24-in. exhaust fan, mounted on an extension of the engine crank shaft, and surrounded by a shroud. Cooling water is circulated by a pump, as is usual in engines of this size.

On a switchboard located between the engine and generator are mounted a volt and ammeter, field rheostat, main switch and overload circuit breaker.

All the elements of control, both of the engine and the electrical system, are within reach of the operator without moving from one position.

Leads are carried from the switchboard fuses to brushes which bear on slip rings mounted on a drum, which in turn is carried on hangers under the right-hand side of the truck body. This drum carries two 210-ft. lengths of 300,000 circular mill flexible cable, any length of which may be unreeled for making taps, and which is rewound by electric power.

Special bronze clamps were designed to make proper electrical contact on any size of pipe between ⅜ in. and 2½ in. by hand nut adjustments and other connections for making contact on hydrants, bib cocks, etc.

All such auxiliaries are designed to be used with the hands alone and without tools. The clamps are connected to the cable terminals by hand nuts.

No details were spared which tend to lessen the time consumed in making connections and preparing for the thawing out operations, as this factor is necessarily much longer than the period of operation of the current.

The melting capacity of the plant is 12 lbs. of ice at 32° F. per minute. This melting effect is, of course, concentrated on the inside wall of the pipe, producing a skin of water surrounding a core of ice which is quickly melted by the flow of the water itself. The initial melting is, therefore, all that is required of the current; and it is found in practice that as soon as a few drops of water show at the delivery point the current may at once be shut off, as the water will complete the work.

Unless the pipes inside the house or building are frozen, it has been found that the best results are obtained when the connection is made as close as possible to the place of entry of the service pipe into the building. The other cable is connected to a fire plug, if one is within 200 ft., or, if not, to a pipe in a neighboring house.

Railways

Principles and Methods for the Valuation of Railroad Property and Other Public Utilities.

(Continued from Page 367.)

DEPRECIATION.

Fundamental Principles.—The corporation, under normal conditions, in addition to an annual return for the use of its capital, is entitled, as already stated, to an allowance sufficient to provide for the net depreciation in the value of all the items of physical property, whether resulting from decay, wear and tear, or other cause, the amount of such depreciation allowance to be sufficient to amortize all such items of property by the time they cease to have value.

A depreciation allowance is, in effect, a payment from the rate-payer to the corporation of a part of its investment, and such part of the investment as has thus been repaid should not appear in subsequent valuations of the property.

The subject of depreciation necessarily deals with somewhat obscure changes which are taking place in the value of property all the time and with the changes which will take place in the future, so that the amount of depreciation actually accruing cannot be accurately determined.

Therefore it is necessary to adopt some practicable and equitable method of determining depreciation allowances which will meet the following requirements:

1. The method should be one which will insure, with as much certainty as possible, payments to the corporation sufficient to amortize the various items of property when they cease to have value.

2. It should so-adjust the annual depreciation allowances as to distribute the burden as equally as practicable among the rate payers of different years.

An Existing View of Depreciation.—There is no subject connected with valuation about which there are more diverse views than those relating to depreciation. Space will not permit the discussion of all of these, but there is one to which the committee desires to call attention before making its own recommendation, namely: the view based upon railroad accounting and practice. From this point of view depreciation, instead of being a "lessening of worth" as defined by the dictionary, is not a lessening of worth for rate-making purposes if the property is kept up to 100 per cent efficiency, although it is recognized that there is a lessening of worth for the purposes of sale, as an old car is not as valuable as a new one. The yearly amount which the corporation is entitled to earn from the rate-payers for depreciation is, with exceptions, the sum necessary for replacing items of property which have become worn out or obsolete, and it is commonly stated that the yearly depreciation is an "operating expense," while the committee has defined it in the previous section as "in effect a payment from the rate-payer to the corporation of a part of its investment."

The water works engineer dealing with long-lived properties, which infrequently require replacement, but come to the end of their lives through inadequacy or obsolescence, recognizes that the allowance for depreciation, if equivalent only to the cost of replacements from year to year, would be wholly inadequate, that is to say, the total amount which would be received from replacements in the first twenty or thirty years would be very insignificant compared with the lessening of worth of the original items of property through the growing inadequacy of some and the tendency toward obsolescence of others.

Although the inadequacy of the depreciation allowances based on cost of replacements are so much more noticeable when applied to long-lived property, it is also true that they are inadequate when applied to railroad properties, although this feature has not been as fully recognized by railroad engineers as by others, on account of prevailing methods of accounting and habits of thought.

The committee believes that its recommendation given below is equitable and practicable and that the method recommended may be applied to both long-lived and short-lived properties. If applied to railroad properties it will give a larger annual allowance for depreciation than is received under prevailing methods, and yet no more than the railroad or other public service corporation is entitled to.

Equal Annual-Payment Method. The committee recommends for practical use what it calls the Equal-Annual-Payment Method of computing depreciation allowances. This method is based on the theory that the most practicable and equitable method is one in which the depreciation allowance for any given item of property, is so adjusted that, taken in connection with the amount which the rate-payer must contribute on account of such item for a fair return on the investment and for repairs and operation, it shall be constant in each year of the life of such item.

TABLE V.—ILLUSTRATION OF EQUAL-ANNUAL-PAYMENT METHOD OF COMPUTING DEPRECIATION.

Assumptions: Property having 20-year life, valued when new at $160. Computations of depreciation allowances based upon 5 per cent. Interest compounded annually. Annual return on capital invested, 5 and 7 per cent.

(1) Age in years.	(2) Value at end of year.	(3) Depreciation during year.	(4) Return on remaining value of property at	(5)	(6) Combined depreciation and return upon investment at	(7)
			5%.	7%.	5%.	7%.
0	$100.00					
		$3.02	$5.00	$7.00	$8.02	$10.02
1	96.98					
		3.18	4.84	6.79	8.02	9.97
2	93.80					
		3.33	4.69	6.57	8.02	9.90
3	90.17					
		3.50	4.52	6.33	8.02	9.93
4	86.97					
		3.68	4 34	6.08	8.02	9.76
5	83.29					
		3.86	4.16	5.83	8.02	9.69
6	79.43					
		4.05	3.97	5.56	8.02	9.61
7	75.38					
		4.26	3.76	5.27	8.02	9.53
8	71.12					
		4.47	3.55	4.98	8.02	9.45
9	66.65					
		4.69	3.33	4.67	8.02	9.36
10	61.96					
		4.93	**3.09**	**4.32**	8.02	9.26
11	57.03					
		5.17	2.85	3.99	8.02	9.16
12	51.86					
		5.43	2.59	3.63	8.02	9.06
13	46.43					
		5.70	2.32	3.25	8.02	8.95
14	40.73					
		5.99	2.03	2.85	8.02	8.84
15	34.74					
		6.29	1.73	2.43	8.02	8.72
16	28.45					
		6.60	1.42	1.99	8.02	8.59
17	21.85					
		6.93	1.09	1.53	8.02	8.46
18	14.92					
		7.28	0.74	1.04	8.02	8.32
19	7.64					
		7.64	0.38	0.53	8.02	8.17
20	0.00					
		$100.00				

In practice it is not feasible to comply exactly with the theory stated, but it can be so nearly complied with that the committee has called the method which it recommends an Equal Annual-Payment Method. This method and every other practicable method which aims to return to the investor the full amount of his investment in each item of property at the time it ceases to have value necessarily requires a determination of the expectation of life of such items.

In order to explain the Equal Annual Payment Method, the accompanying Table V is presented, but the table takes into account only the two principal of the three factors above mentioned, namely, (a) the annual depreciation allowance, (b) the return for the use of capital, and omits (c) the minor effect of the cost of repairs and operation which will be considered subsequently.

The illustration is based upon an item of property valued when new at $100 and having a twenty-year life. The depreciation during each year is based on interest at 5 per cent per annum, compounded annually, and for purposes of illustration the annual return for the use of the capital invested is reckoned at both 5 and 7 per cent.

The depreciation during each year as given in Column 3 of the table, is exactly the same as if computed by the so-called Sinking-Fund Method; that is, it is the constant annual payment to a sinking fund added to the year's interest on the accumulated fund at the beginning of the year.

When the annual return for the use of the capital invested is based upon the same percentage that is used in computing the depreciation, the method provides exactly the same total payments on account of depreciation and return combined as would be obtained by establishing a sinking found which earned 5 per cent interest, but it does not contemplate the maintenance of such a fund. Thus, at 5 per cent the annual contribution to a sinking fund to wipe out $100 of value in twenty years is $3.02, which added to 5 per cent of the $100 originally invested in the item of property, gives a total of $8.02 for each year.

It will be seen by reference to Column 6 of the table that $8.02 is the combined annual depreciation and return for each year of the twenty by the proposed method, but this sum in the table is made up in a different way. The annual return, instead of being based upon the full value of the property, is based upon its net or depreciated value at the beginning of each year, as given in Column 2, such net value being obtained by subtracting successively the depreciation during each year. For example, the depreciation during the first year is $3.02, which subtracted from the full value leaves $96.98 as the depreciated value at the end of the first year; deducting from this figure the depreciation during the second year, amounting to $3.18, leaves $93.80 as the depreciated value at the end of the second year, and so on.

The return for the use of capital in the third year would be 5 per cent on $93.80, the depreciated value at the beginning of the year—equal to $4.69 as stated in Column 4, which, added to the $3.33 for depreciation during the year, gives $8.02, as stated in Column 6.

Although the Sinking-Fund and Equal-Annual-Payment Methods, under the assumptions made, provide for the same annual rates, there is a difference in the principle involved. The Sinking Fund Method provides for a uniform annual payment to the sinking fund which must be invested and earn interest in order that the accumulations may amount to the full value of the item of property at the end of its life, while the yearly depreciation allowances by the Equal-Annual-Payment Method include sums equivalent to the interest in the Sinking-Fund Method and therefore amount without interest to such full value. Thus, referring to the third column of the table, the depreciation during the second year is $3.18, which is equivalent to $3.02, the depreciation during the first year with 5 per cent interest added. The depreciation during the third year is $3.33, equal to $3.18 with 5 per cent interest added, and the same rule applies to the end of the column.

The Equal-Annual Payment Method distinguishes plainly between the return for use of capital, available for dividends, and the depreciation allowance which should be credited to the capital account; thus in the tenth year of the item of property illustrated in the table, the return available for dividends is $3.09, as given in Column 4, while the depreciation allowance to be credited to capital is $4.93, as given in Column 3.

The method is an advantageous one for valuing an old property because the rates remain the same regardless of the ages of the several items of property.

Attention may be called to another feature illustrated by the table. If the item of property having a 20-year life has existed for say 12 years, its depreciated value at the end of the year, as given in Column 2, is $51.86, and the depreciation allowance for the 13th year is $5.43. This allowance is the same amount as an annual sinking-fund payment on a new item of property having a value of $51.86 and a life of eight years.

The foregoing statements have been based on the use of 5 per cent in all computations, but there are reasons, to be referred to subsequently, why the rate of return for the use of capital will generally be larger than the interest rate used in computing depreciation, and when there is such a difference in the rate, the combined annual depreciation and return for the use of capital, as given in Column 7 of the table, will not be strictly uniform from the beginning to the end of the 20-year period.

It has already been stated that in preparing the table only two of the three factors were taken into account, and the introduction of the third factor—the cost of repairs and operation—will affect the degree of uniformity because as a rule such cost increases as the property grows older. The inclusion of such cost, therefore, will increase the quantities in Columns 6 and 7 to a greater extent in the later than in the earlier years, making the results based on Column 6 somewhat greater in the later than in the earlier years, and those based on Column 7, under most circumstances, more nearly uniform than they are in the table.

It is impracticable to devise any rule which will provide for strictly equal annual payments if the cost of repairs and operation is included, because while as a rule such cost is a small factor in the problem it is variable, and there are exceptional cases in which the cost of repairs and operation differs greatly from the beginning to the end of the life. Practically, however, the exceptional conditions mentioned are unimportant, because where the several items of property have different lives there will be a tendency to equalization of payment on account of the three items of return on capital, depreciation, and repairs and operation when the whole property is considered together.

Rates of Interest and Return.—It is the judgment of the committee that a smaller percentage should be used for computing the annual depreciation allowances than would be a proper rate of return for the use of capital invested in public service properties.

In the past many of those who have discussed the subject, having in mind the creation of depreciation or sinking funds, have adopted a low rate of interest in making their computations of depreciation on the ground that funds safely invested would not give a net return of more than 3 or 4 per cent.

It is the view of the committee that no such funds should be created, except temporarily, and that the rate of interest to be used in computing depreciation allowances should be substantially the rate at which the corporation can borrow money upon adequate security. In recent years this rate has been about 5 to 5½ per cent for the average well-established corporation. To adopt a rate much above 5 per cent for computing depreciation allowances would result in smaller allowances for depreciation in the early years of the life of each item of property, especially long-lived property, than the committee believes to be advisable.

The rate of return upon the investment should be higher than that used in computing the depreciation allowance, so as to cover interest and profit, and thus give a return commensurate with the risks involved in such investments.

While, as stated above, well-established corporations have been able in recent years to borrow money at rates of 5 to 5½ per cent, in practice not more than from one-half to three-quarters of the capital required can be raised at these rates, higher rates having to be paid upon the rest of the capital required by reason of the less adequate security.

Investment of Money Received by the Corporation for Depreciation Allowances.—Assuming the committee's view to be correct, that a depreciation allowance is a return to the corporation of a part of its investment in existing items of property, the money so returned should be treated as a part of the capital and used for any purpose for which the corporation is authorized to use capital, namely: for additions, replacements, betterments, or the extinguishment of outstanding obligations.

It may happen that the amount earned for depreciation allowances will be in excess of immediate needs, and in such cases the accumulations become a part of the working capital until there is an opportunity to invest them in the property.

Depreciation Allowances an Important Feature in Rate Regulation.—In any equitable and comprehensive regulation of rates it is necessary that the amount of the depreciation allowances should be determined by logical and proper methods uniformly applied from year to year.

In the case of some properties made up of items which have a short life, like those of telephone companies, the amount which the corporation is entitled to earn for depreciation may be as great as the amount which it is entitled to earn as a return upon its capital.

Where the property is made up of long-lived items, as in the case of water works property, the amount of the depreciation allowance is still important and should be carefully determined by public service commissions and engineers if justice is to be done to both the corporation and the rate-payer.

Depreciation allowances should be such that the corporation shall have received, when its property is worn out or become obsolete, neither 50 nor 150 per cent of its value, but as nearly 100 per cent as the best methods of determining the depreciation allowances will permit.

Expectation of Life as a Basis for Estimating Depreciation.—It is generally admitted that the owner of a public service property is entitled to earn annually a sufficient sum for depreciation to keep his investment unimpaired, and this view is supported by the Knoxville decision of the United States Supreme Court, which stated that the company "is entitled to see that from earnings the value of the property invested is kept unimpaired, so that at the end of any given term of years the original investment remains as it was at the beginning."

Notwithstanding the adoption of this view, many have objected strongly to using the expectation of life of the different items of property as a basis for determining the depreciation of such property, claiming, especially in the case of property subject to functional depreciation, that any attempt to estimate the probable life is purely guesswork.

There cannot be any doubt that many parts of a public service property which will not fail through wear and tear or decay are becoming less valuable each year because of approaching inadequacy or obsolescence, and it is the judgment of the committee that the corporation is entitled to a depreciation allowance corresponding to the lessening worth of all its property. To omit functional depreciation would not, under the present rulings of the courts, do justice to the corporation, because if the full value of the property has not been returned in the form of depreciation allowances while the property is in use, it will afterward be excluded from the valuation as unused and obsolete property.

In other words, the corporation should receive the whole 100 per cent of the value of an item of property during its lifetime because there is little likelihood that it will receive any subsequent allowance on account of such property, and this can be done only on the basis of the expectation of life of all items of property.

It is not claimed that one can determine accurately the life of any given structure or item of property, but that if experience is used as a guide inaccuracies in such determination will, to a considerable extent, balance one another so as to give a fairly correct result in regard to the plant as a whole.

It is obvious that precision cannot be attained when one is dealing with the future, but in the judgment of the committee a greater degree of precision can be attained by a method which deals with the expectation of life of items of property than in any other way.

The statistics of the actual life of public service property of a permanent character gathered by the committee show that such actual life is much shorter than the estimates generally given in publications on the valuation of public utilities, and probably much shorter than the builders of the structures anticipated. Changes which cannot be foreseen are taking place all the time, and the life assigned to the various items of property should be based largely on experience rather than on optimistic views of the future.

Sinking-Fund Method.—The Sinking-Fund Method of computing depreciation allowances, as already stated, gives the same results as the Equal-Annual Payment Method, but the use of the words Sinking Fund appears to have misled some courts and commissions so that they have objected to the depreciation allowances so computed on the ground that the method involved the creation of a sinking fund to be kept intact until the replacement of the property at a later time. (See Decisions of Courts and Commissions in Regard to Depreciation.)

Straight-Line Method. The Straight-Line Method of computing depreciation assumes that the value of the item of property under consideration decreases an equal amount in each year of its life; that is, the depreciation in each year is the full value of the property divided by the number of years of its life. The method is simple and may appear to some to provide for equal annual payments by the rate payers, but instead it requires a much larger payment in the earlier years of the life of the item of property than in the later years, as will be seen by the illustration furnished by Table VI.

TABLE VI.—ILLUSTRATION OF STRAIGHT-LINE METHOD OF COMPUTING DEPRECIATION.

Assumptions: Property having 20-year life valued when new at $100. Annual return on capital invested, 5 and 7 per cent.

(1) Age in years.	(2) Value at end of year.	(3) Depreciation during year.	(4) Return on remaining value of property at 5%.	(5) Return on remaining value of property at 7%.	(6) Combined depreciation and return upon investment at 5%.	(7) Combined depreciation and return upon investment at 7%.
0	$100.00					
1	95.00	$5.00	$5.00	$7.00	$10.00	$12.00
2	90.00	5.00	4.75	6.65	9.75	11.65
3	85.00	5.00	4.50	6.30	9.50	11.30
4	80.00	5.00	4.25	5.95	9.25	10.95
5	75.00	5.00	4.00	5.60	9.00	10.60
6	70.00	5.00	3.75	5.25	8.75	10.25
7	65.00	5.00	3.50	4.90	8.50	9.90
8	60.00	5.00	3.25	4.55	8.25	9.55
9	55.00	5.00	3.00	4.20	8.00	9.20
10	50.00	5.00	2.75	3.85	7.75	8.85
11	45.00	5.00	2.50	3.50	7.50	8.50
12	40.00	5.00	2.25	3.15	7.25	8.15
13	35.00	5.00	2.00	2.80	7.00	7.80
14	30.00	5.00	1.75	2.45	6.75	7.45
15	25.00	5.00	1.50	2.10	6.50	7.10
16	20.00	5.00	1.25	1.75	6.25	6.75
17	15.00	5.00	1.00	1.40	6.00	6.40
18	10.00	5.00	0.75	1.05	5.75	6.05
19	5.00	5.00	0.50	0.70	5.50	5.70
20	0.00	5.00	0.25	0.35	5.25	[illegible]
		$100.00				

This table differs from Table V only in the method adopted for determining the amount of depreciation. Columns 6 and 7 in both of these tables are representative of the rates which the public should pay as determined by the two methods. In the Table V based upon the Equal Annual Payment Method the rates are nearly equal in each of the 20 years, while in Table VI based upon the Straight-Line Method they are about twice as large when the property is new as in the last year of its life.

This illustration shows that in the case of those properties consisting largely of long-lived structures, in which the annual cost of renewals is very much greater in the later than in the earlier years of life, the Straight-Line Method is undesirable for the following reasons:

(1) That it is inequitable as it makes a much greater demand on the rate-payers of the earlier years than on those of later years.

(2) That it permits the corporation to earn for depreciation in the earlier years a larger sum than is necessary to maintain the full value of its investment.

(3) The method requires higher rates dur-

ing the early part of the life of a property when it is more difficult to pay such rates than in the later years.

The objections to this method are less in the case of short-lived property, as will be seen by the following illustration based on 10-and 40-year life, respectively, instead of 20. A 7 per cent annual return has been assumed and the results are comparable with those in Column 7 of Table VI.

	10-year life.	40-year life.
	$17.00	$9.50
Last-year rates	10.70	2.67

Actual-Inspection Method.—In the valuation of property the amount of depreciation has frequently been determined without reference to any fixed rule, the appraiser inspecting the property and using his judgment as to the amount of depreciation. In such cases the judgment is often based almost wholly upon the observed physical depreciation of the property. Earthwork, substantial masonry, and other property in excellent condition are appraised at or near their full value. Such appraisals are likely to result in a much smaller total depreciation in the value of the property than when based upon the expectation of life of the different items of such property and, as a consequence, would logically entitle the corporation to a much smaller annual allowance for depreciation than when a method is used which includes depreciation of all kinds.

Under such a method many items of property would become obsolete before the corporation had received an amount for depreciation approximating 100 per cent of their value.

There is a fundamental objection to depending upon the personal judgment of appraisers, as this method determines only the total depreciation of the property and not the amount of the depreciation allowance which the corporation is entitled to earn, unless it is to be assumed that the difference in the amount at which the property is appraised from time to time furnishes a measure of the amount of depreciation which should be allowed during the period between such appraisals. It is obvious that very erratic results would be obtained by such a method, especially if different persons acted as appraisers in different years.

This method does not take account, to any considerable extent, of functional depreciation, and the great injustice which may be done a corporation by the failure to take such depreciation into account has already been stated.

It is not intended by this condemnation of the Actual-Inspection Method to indicate that there should not be an actual inspection of all property in connection with other methods of determining the amount of depreciation. Such inspection should be made partly for the purpose of noting the condition of the property and partly for the purpose of determining the probable future life of the various items of property.

Replacement Method.—It has sometimes been claimed that, when a property is maintained in good condition by repairs and by making replacements when necessary, rates should be based upon the full value of the property. Under this theory, the corporation would be entitled to earn as a depreciation allowance the amount actually expended for replacements. With an unregulated and short-lived property, this method has in the past produced fairly satisfactory results when a property has been maintained in excellent condition by charging many betterments as well as replacements to current expenses. It has no place in any proper theory of rate regulation, especially in the case of long-lived property, as great injustice may be done by the application of the method. It holds back from the corporation an annual sum for depreciation which the United States Supreme Court in its Knoxville decision says the corporation is entitled to earn to keep the value of the property unimpaired. If the money justly due the corporation annually is not then paid it is not likely that it will be paid subsequently.

Application of Equal-Annual-Payment Method.—In a strict sense, the term "item of property" which has been frequently used in this report, may be so defined as to cover insignificant items, such as, for instance, small tools which may have a life of one, two or three years.

It is obviously impracticable in any system of accounts to consider the depreciation from year to year of such short-lived property, and its cost should be considered a part of the current expenses, except in cases where a temporary charge is made to the account of stock on hand.

In determining the expectation of life of a given item of property, one necessarily deals with such item taken as a whole. For instance, a wooden building may be assumed to have a life of thirty years. During its life it is necessary many times to replace the paint. Such replacements during the life of a building are not taken care of in the depreciation allowance herein recommended and should be charged to current expenses. The same rule applies in a multitude of cases, such as the replacement of bolts in a railroad track, the renewal of parts of locomotives or pumping engines, the placing of additional ballast on a railway road-bed, and to much greater expenditures, as, for instance, the replacements of parts of road-beds or machines after washouts or accidents.

Should the method herein recommended of providing for depreciation be adopted, the decision as to where the line shall be drawn between the low cost and shorter-lived items of property on the one hand—the depreciation of which can best be taken care of by charging the cost of replacements to current expenses—and the high cost and longer-lived items of property on the other hand—the depreciation of which can best be taken care of on the basis of the expectation of life—may well be left to the Public Service Commissions and others having charge of the valuation of the different kinds of public service property.

As a general statement, it may be said that the basis of expectation of life should be used for all important items of property having a life of more than, say, five years, except where it may involve unwarranted expense for accounting or be impracticable to separate what would properly be, under the Equal-Annual-Payment Method, charges to current expenses and to capital. For instance, it would not be practicable to separate the charges for labor for tamping railroad ties that are renewed from that for tamping ties already in use, although if it were desired to avoid entirely the use of the replacement method of determining depreciation, it would be feasible to estimate what part of the total yearly charge for such tamping is attributable to the work on new ties and therefore properly chargeable under the Equal-Annual-Payment Method to capital, and to transfer the estimated sum annually from the operating to the capital account.

Under a continuous system of accounting small items of property purchased and charged directly to current expenses should obviously not be included in the valuation of the property upon which the annual return is to be based.

Grouping Items of Property.—In many cases where a valuation is made, there are certain kinds of property, such as cars, rails, ties, telephones and telephone poles, which exist in large numbers and are of all ages; these can be dealt with by grouping them.

If there be a group consisting of a large number of items, each having the same expectation of life and existing in equal numbers of each age, the annual depreciation allowance for the group, whether computed by the Straight-Line Method or the Equal-Annual-Payment Method, would be the same. There is, however, a difference in the average remaining value of such a group of items on which the annual return is to be made. If the Straight-Line Method be applied on the basis adopted in computing Table VI, using the value at the beginning of a year as the basis for computing the return, the average remaining value will be 52.5 per cent of the value new, while if the Equal-Annual-Payment Method be applied on the basis adopted for computing Table V, the average remaining value will be 60.5 per cent. If instead of computing the return upon the remaining value at the beginning of each year it were computed upon such value at the middle of each year, the average remaining value would be, respectively, 50 and 57.6 per cent.

Scrap Value.—All of the foregoing statements regarding depreciation relate to property which has a negligible scrap value. In the case of items of property like steel rails, which have a large scrap value, the statements are not strictly applicable, but the same general principles are involved; the amount of depreciation to be provided for being the difference between the new and the scrap value.

Decisions of Courts and Commissions in Regard to Depreciation.—There were some court decisions made from 10 to 20 years ago rejecting the claims of corporations for annual allowances for depreciation. The decision of the United States Supreme Court in the Knoxville case in 1909, states clearly, however, that the corporation is entitled to earn an annual depreciation allowance, and that the return should be based on the depreciated value of the property. This decision has been upheld by many subsequent decisions of the same court.

The portion of the decision relating to depreciation is as follows:[*]

> The cost of reproduction is one way of ascertaining the present value of a plant like that of a water company, but that test would lead to obviously incorrect results if the cost of reproduction is not diminished by the depreciation which has come from age and use.
>
> The cost of reproduction is not always a fair measure of the present value of a plant which has been in use for many years. The items composing the plant depreciate in value from year to year in a varying degree. Some pieces of property, like real estate, for instance, depreciate not at all, and sometimes, on the other hand, appreciate in value. But the reservoirs, the mains, the service pipes, structures upon real estate, standpipes, pumps, boilers, meters, tools and appliances of every kind begin to depreciate with more or less rapidity from the moment of their first use. It is not easy to fix at any given time the amount of depreciation of a plant whose component parts are of different ages, with different expectations of life. But it is clear that some substantial allowance for depreciation ought to have been made in this case.
>
> A water plant, with all its additions, begins to depreciate in value from the moment of its use. * * * the company is entitled to earn a sufficient sum annually to provide not only for current repairs, but for making good the depreciation and replacing the parts of the property when they come to the end of their life. The company is not bound to see its property gradually waste, without making provision out of earnings for its replacement. It is entitled to see that from earnings the value of the property invested is kept unimpaired, so that, at the end of any given term of years, the original investment remains as it was at the beginning.
>
> If, however, a company fails to perform this plain duty and to exact sufficient returns to keep the investment unimpaired, whether this is the result of unwarranted dividends upon over issues of securities, or of omissions to exact proper prices for the output, the fault is its own. When, therefore, a public regulation of its prices comes under question, the true value of the property then employed for the purpose of earning a return cannot be enhanced by a consideration of the errors in management which have been committed in the past.

The Equal-Annual-Payment Method accords with the principles laid down in this decision, while any method which does not make provision for depreciation as it accrues would not be in accord with such principles.

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[*] Knoxville v. Knoxville Water Co., 212 U. S., 1, January 4, 1909.

n this case the court was dealing with an esablished normal plant. It cannot be supposed that the same reasoning would be applied to a new plant unable in its early years o earn a sufficient sum to cover depreciation nor to one which immediately after important additions had been made was temporarily unable to earn such sufficient sum. In the judgment of the committee the decision of the court in the Knoxville case with regard to depreciation is equitable.

Except for the basic principles laid down in the Knoxville decision, the decisions of courts and commissions in regard to depreciation have been very variable. The Straight-Line Method has been used and courts have approved valuations in which the depreciation has been computed by this method. They have, however, more often approved valuations in which the sinking fund, or as the committee has termed it, the Equal-Annual-Payment Method, has been used.

In some cases where both the Straight-Line and the Sinking-Fund Methods have been presented, the Sinking-Fund Method has been rejected, but this appears to have been the result of the assumption that the Sinking-Fund Method, instead of being a practical method for fixing the rate of depreciation, is a method for creating a sinking fund to pay for the replacement of the plant at a later date.

For instance, in the Louisville, Ky., telephone rate case, District Judge Evans, in fixing the annual depreciation allowance makes this statement:*

> Of course, our estimate could not be based upon the proposition that the per centum set apart to cover depreciation would be deposited in bank or loaned out from year to year so as to accumulate and be on hand at the end of 14 years, and to be then used to construct an entirely new plant, and so on from period to period. In such a case the public would not only have a service that would progressively grow worse until its operations ceased altogether, but it would thereafter get no service at all until a new plant replacing the old could be completed and put into operation. The question rather has been, What does experience show to be the proper average per cent of annual earnings which the company should expend in order to insure that its plant at the end of 14 years will be as good as it now is, and in the meantime render to the public that good service which its duty to the public requires?

There was a decision against a Sinking-Fund Method in a New York tax case in 1911. In this case, Judge Gray of the New York Court of Appeals states (at page 235):†

> The courts below determined that the relator was entitled to make annual depreciation charges, amounting in the case of the borough of Manhattan to the sum of $360,613.65 and in the case of the borough of The Bronx to the sum of $37,435.67, for the purpose of creating a fund to provide for the depreciation of its various properties; upon which interest at four per cent, compounded, would produce a sum, at the termination of the ascertained physical life of the several classes of property, equal to the cost of the particular property. While I am personally, of the opinion that the creation of such an amortization fund furnishes the best rule for adoption in such a case as this, in working out the value of special franchises, the majority of my brethren entertain a different view. They think that the annual allowance for depreciation should be computed by dividing the values of the various kinds of tangible property by the number of years of their respective estimated physical lives and that will be the opinion of the court.

Judge Haight, of the same court, in a concurring opinion, objects to the Sinking-Fund plan as follows (at page 239):‡

> The Special Term in this case, however, adopted a plan of amortization upon which an annual sum was authorized to be set apart as a sinking fund, which, by compounding the interest thereon for a period equal to the life of the structure, tracks, engines, machinery and rolling stock, would at the end of that period create a fund sufficient to replace the property. The difficulty with such holding is that railroad corporations do not reconstruct their railroads and rolling stock in that way. In order to afford proper protection to the public they are required to maintain a high state of efficiency both in roadbed and rolling stock. The relator's railroad has been in existence already for about thirty years and some portion of its property has already suffered from decay and use to such an extent that portions thereof have to be reconstructed and made new each year. Old ties have to be removed and replaced with new ones; old rails that have become worn and battered have to be removed and their places supplied with new rails, and so the work of reconstruction progresses from year to year. It is not the waiting forty or sixty years to reconstruct, during which time the amount set apart as a sinking fund may be doubled many times over by compounding the interest, but it is the annual expenditure for reconstruction which is to be paid for at the time that the construction is made. To illustrate: Suppose the average life of the tangible property of a railroad, outside of the land itself, to be sixty years and the cost of reconstruction to be sixty million dollars, it would follow that one million dollars would have to be used each year in reconstruction, and that amount would have to be annually used for that purpose; but under the plan adopted in this case, instead of deducting from the gross earnings the amount necessarily expended for that purpose, a small fraction of that sum, viz., $4,200, only is allowed to be deducted, a sum which, with the interest compounded for the next sixty years, would amount to a million dollars. Under such a plan the company would be practically prohibited from annually constructing a portion of its road and thus prevented from keeping it in that state of efficiency which the public demands. Of course, the necessities of reconstruction vary from year to year; some years it may be greater than others, but the assessors each year can easily ascertain the sum required for that purpose. I think, therefore, that we should adhere to the rule sanctioned in the *Jamaica* case, and that a gross sum should be deducted annually for the purposes of reconstruction.

It seems clear that in these opinions the opposition to the Sinking-Fund Method is directed against the establishing of a sinking fund to be kept intact for a long series of years, and not against the method on account of its providing for uniform annual payments.

The illustration given in Judge Haight's concurring opinion seems to indicate that he thought the Sinking-Fund Method would give an annual allowance for depreciation of only a small fraction of the sum determined by the Straight-Line Method. The property in question was that of the Manhattan Railway Co., and in view of the age of the structures and the varying ages of the equipment it seems not unlikely that the annual allowance for depreciation would have been as great by the Sinking-Fund as by the Straight-Line Method of measuring depreciation.

The Sinking Fund Method of measuring depreciation has been approved by the Wisconsin Railroad Commission and has been rejected by the New York Public Service Commission, First District. The decision in the latter case is open to about the same comment as the decision in the above cited court cases, as will be seen by the following quotation (page 317, Whitten):

> Without going into the technical details of these two plans, suffice it to say that the fundamental difference is that the former [Straight-Line Method] assumes the property to decrease uniformly in value from year to year, the latter [Sinking Fund Method] that it follows a parabolic curve. The former assumes that the decrease will be met year by year as it occurs, that the payment from earnings will be immediately expended and that it will not accumulate at compound interest. The latter assumes that nothing will be spent before the end of the period, that it will all accumulate and that impairment of capital need not necessarily be met as it occurs.

There are other decisions relating to physical and functional depreciation which do not accord with one another. Upon the whole, it may be said that the court and commission decisions have not reached that degree of definiteness which will prevent the application of the method herein recommended.

A Fallacy in the Discussion of Depreciation. At the risk of repeating what has already been stated, the committee will refer again to the fallacy of the view expressed by engineers of much experience in valuation, that rates based upon the depreciated value of a property are necessarily lower than those based upon its full value.

A recent work on Valuation* gives these illustrations (pages 207-208):

> Assume that the life of a large part of a complete property is 20 years, then at the end of 19 years and 6 months, if theoretical depreciation is considered, the present value of the property would be small and the rates based thereon would include nothing in the way of return on the large part of the property, still 100 per cent useful. A year thereafter the property being entirely replaced and new, the rates would be incomparably higher.
>
> Consider two surface railways running out parallel avenues from the center of a city to the suburbs, both alike in construction, but one 10 years old and the other put in operation within a year. If theoretical depreciation is considered the present values of these two properties are quite different, the older road being worth appreciably less than the new road, although the original cost of installation may have been the same in both cases. Under such circumstances, is the older road to be allowed to charge only a four-cent fare, assuming that that gives a fair return on the estimated present value, while the new road must charge a five-cent fare for the same return on its estimated value? What would be the result practically of such method of fixing rates? The old road would be swamped with business and the new road would be unable to maintain its earnings.

The "theoretical depreciation" referred to in these quotations is the depreciation based upon the length of life of the property. It has already been shown in this report that the life of the property used in connection with the Equal-Annual-Payment Method of determining depreciation allowances results in practically uniform rates whether the property be old or new. There is, therefore, no logical ground for the statements above quoted or for the following quotation based upon them:

> Can such fanciful and variable bases be intended by the Supreme Court to be taken as that on which rates are to be estimated and regulated? Such conclusion would be illogical, unreasonable and unfair.

In a recent discussion† of the paper on the "Physical Valuation of Railroads," by William J. Wilgus, M. Am. Soc. C. E., not only have several of those taking part in the discussion favored the use of the full value of a property as the basis for rates, but some have endorsed the above mentioned fallacy. One writer says:

> A new public utility company starting today would be entitled to a fair return on its actual investment, and it would lead to "confusion worse confounded" were a part of that investment "written off" each year because of depreciation. The thing could be done but no useful purpose would be served by doing it, while, on the contrary, rates of charge for service would fluctuate with an ever-changing depreciated value. There is a good deal of "horse-sense" in the illustration used by an attorney who argued that the price of milk is not an indication of the age of the cow, growing less as the cow grows older, until it would be almost given away in the latter days of the cow's life.

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*Cumberland Telephone and Telegraph Co. v. City of Louisville, 187 Fed., 637, April 27, 1911.

†People ex rel. Manhattan Railway Company v. Woodoury, 203 N. Y., 231, October 17, 1911.

‡People ex rel. Manhattan Railway Company v. Woodbury, 203 N. Y., 231, October 17, 1911.

*"Valuation of Public Utility Properties," by Henry Floy, New York, 1912.

†Proceedings of the American Society of Civil Engineers, October, 1913.

> What the provident owner of a cow does is to charge a price for milk which will give a fair return on the investment (not on the depreciated value) plus enough to cover "operating expenses," including depreciation.

Another writer says:

> In the case of a public utility, adequately maintained and rendering efficient service to the public, to deduct for accrued depreciation according to the practice of some of our theorists, necessarily results in confiscation of investment.
>
> Certain appraisers of reputation have taken the ground that if, for instance, railroad ties have an effective life of 10 years, and one-tenth of the total number is renewed each year, there is no need for a depreciation reserve, but the value of these ties as a whole should be depreciated 50 per cent. Nothing, according to the writer's way of thinking, could be more fallacious. The property, as far as this item is concerned, is thus maintained at maximum efficiency. What more could be done? Why then should one-half of this portion of the investment be confiscated?

It will be observed that the writers in all these cases ignore the fact that a depreciation allowance earned from the rate-payers is a return of a part of the investment, and that after that part of the investment is returned, it is neither inequitable nor confiscatory to base future returns upon the depreciated value, which is the remaining investment in the property. They also erroneously assume that the rates necessarily decrease when the depreciated value is used.

It is true that of the return on a given item of property, the part applicable to dividends becomes less as the property grows older, but this is as it should be; a part of the capital invested in that item has been returned and has either been invested in additions or replacements, or temporarily in a depreciation fund, so that the whole of the original investment is at all times earning returns applicable to dividends.

Past Practice Has Underestimated Depreciation.—Ideas in regard to depreciation have in the past been based very largely upon practical methods of railroad accounting and not upon a consideration of the fundamental principles upon which depreciation allowances should be based, and even where the proper principles have been taken into account, the tables prepared by engineers of the expectation of life of different components of public service works have been based mainly upon physical depreciation, without making due allowance for the shortening of life caused by inadequacy and obsolescence.

Your committee believes that there have been many cases in the past in which the annual allowances for depreciation have been under-estimated, and that consequently engineers, public service commissioners and the courts should give especial attention to the amount of the annual depreciation allowances, so as to make them conform more nearly with the facts.

The inter-relation in rate-making cases between the annual depreciation allowance and the depreciated value of the property should never be lost sight of.

(To be continued.)

Snow Removal Conference.—The public is becoming more and more insistent that any considerable fall of snow not only be removed from the highways, but removed quickly. The cost of snow removal work in cities has been going up rapidly, but the methods used have not materially improved. With the co-operation of the Commissioners of Public Works of New York, Boston, Pittsburgh, Washington and other cities, the Department of Public Works of Philadelphia has called a "snow removal conference" to be held in Philadelphia, April 16 and 17. At this conference the effort will be made to have the various factors affecting the cost and efficiency of snow removal thoroughly discussed. Any one interested in the subject will be welcome to attend. It is hoped that out of this conference will come some statement that will afford a basis for formulating an intelligent policy as to snow removal for any given locality.

Modern Methods of Economical Location.

To the Editors: The writer has read with interest the article on "Modern Methods of Economical Location" in Engineering & Contracting and he agrees in general with the author that more money could be spent to advantage in preliminary work than is done at present.

He is exceedingly skeptical, however, regarding the value of some of the refinements suggested; especially do the charts to show future traffic for ten, twenty, thirty and forty year periods, seem of doubtful utility. Such charts would reflect more or less the temperaments of their makers and a comparison of two of them made as suggested, one by a traffic man of sanguine temperament interested in the success of the projected road, and the other by a passenger traffic manager trying to keep his road off the shoals while waiting for a possible five per cent advance in freight rates, would show such a startling difference that as engineering data they would be considerably harder to reconcile than the contour lines of Smith and Jones taken over the same country from different lines of levels. Even could the personal equation be eliminated there are too many factors, impossible to foresee, which enter into the development of a territory to make such charts of practical value. One live booster with capital can place a community far beyond the limits which could have been prophesied by a survey of the agricultural and commercial features before his advent. On the other hand competition can come at any time and plans based on the most carefully and conservatively made plans would go far wrong. There are a dozen other contingencies which could make the estimate of the optimist way too low or that of the pessimist as much too high.

Railroads are today built better than they were in the past but a dollar to the average promoter still looks as large as a cart wheel and he would prefer to save on construction the fifty cents which he has rather than to save or make three or four dollars in the future. The remedy for this, if it needs a remedy, will come from better financial methods rather than from better locations and the method of striking a balance between present traffic possibilities and the physical characteristics of the country does not serve as badly as the author seems to suppose. To spend much additional money on location or construction for the sake of possible traffic forty, or even twenty years, in the future would seem to the writer very much the same as for a five-engine railroad to put up a ten-stall concrete roundhouse because the price per yard of concrete would be a dollar or so less in the larger structure.

The railroads with "grades following the grass roots and alignment the local subsidies" have built up the United States. Some of them have served their communities well. Reconstructed they have become part of our transcontinental systems and they have justified their existence for it was either a railroad with such grades and alignment or no railroad. Others have proved exceedingly disastrous to their promoters and are operated at a loss today, years after their completion but, for the most part, it was not poor engineering or lack of broad-minded managers as the author seems to think, that was responsible for their construction. It was the same lack of knowledge that obtains today as to where and to what extent future development will take place. The writer knows of a railroad, in a part of California as fertile as any in the state, which is doing no more business now than it did upon its completion thirty years ago. This road is crooked, both horizontally and vertically, but it is cheaply built and has been a fair investment and given good service. The prospects for the country it serves were as good, when it was built, as any in the State, but the tide of progress has gone above and below it and its time has not yet come—and it has not been holding the bag for 30 years.

The author seems inclined to criticise past railroad location, basing his criticism on data obtained from contour maps alone. Rock cuts, sliding ground and sink holes are excellent excuses for deviation from what would appear, on the map, to be the best location and many a trunk line is a patch work of parts of small independent lines each of which was built to serve a different purpose than that for which it is now used. It is not as efficient, surely, as an ideally located and constructed road would be but such roads cost money and money is not always available. Meanwhile this road does the business and when money is forthcoming, grades and alignment can be revised and bad stretches abandoned. It is generally in a new country easier and cheaper to build new than reconstruct and there is more difference between a five-tenths and a one per cent location than a rise or fall of 26.4 ft. to the mile.

Plaster casts undoubtedly have a substantial value in such engineer works as locks and dams but the writer questions their value in railroad location and he seriously doubts the possibilities of chief engineers being able to make a practice of satisfactorily settling on townsite locations to suit the convenience of the operating department in making meeting points for future trains, devoutly as that consummation is to be wished.

The system to be followed in the preliminary survey will always remain a matter of individual judgment but there is much to be said in favor of the old method of "feeling the country out" even if "miles and miles of expensive chopping through the woods" is required and it will not do to condemn, as of small caliber, the engineers, whether college bred or self educated teamsters who have located and are still locating by that method eighty per cent of the railroads which traverse the continent.

Yours truly,

H. G. Butler.

564 61st Street, Oakland, Cal., March 2, 1914.

(A copy of the above letter was submitted for comment to the author of the article criticized and his reply is given below.—Editors.)

To the Editors: The article was written to draw forth healthy criticism, and thus help to develop a scientific system of railroad location.

There are two portions of the United States to which railroad traffic diagrams evidently could not have been applied. These are southern California and Florida, places whose original cash assets were climate and boosters. This combination of sunshine and "wind" has drawn from other portions of the United States a railroad traffic that never could have been foreseen, and the end is probably not yet. Boosters and boosting have changed things some, but the vast bulk of the continent has been and still is developing along the lines of natural resources. Boosting never started rice farming without the water, coal mining without the coal. There are certain industries in belts, zones and limits of longitude and otherwise, the real bona fide natural consequences of soil, water and minerals, that are beyond the power of man to boost or control, and these natural conditions and their developments are things that could have been foreseen and their traffic arranged for, had promoters been willing to pay for the services of truly scientific investigators. Biased or easily influenced investigators are not wanted to tabulate diagrams as shown in the original article of Feb. 18. Facts and figures of growth and development are what are needed, and from these the probabilities of the future are to be forecast. This is done in almost every business but railroad engineering, and it can be done there, and is being done more every year.

Mr. Butler very properly does not name the railroad in California he has in mind, but it may not be amiss to suggest the California & Nevada R. R. as a case in point. This road was built by D. O. Mills in the early seventies, and originally ran from Reno, Nev., down in behind the Sierre Nevada mountains in California, away from the stream of industry and settlement, though in as fertile a country as any. But its geographical location was against it to start with—a range of mountains on the west and a desert on the east—and in later years it has been drained and robbed of its waters to make the Los Angeles country a success. Its isolation could have been anticipated and shown on diagrams, but the huge waterway several hundred miles in length is something no one could have foreseen. As late as 1908, when the engineers were investigating it for a broad gage, they found the country still in a Rip Van Winkle sleep. Its inhabitants were energetic and industrious, but they had no market for their produce. The country roads were lined for miles with fine English walnut trees, the nuts rotting on the ground season after season, and there were miles of melon fields, the melons never shipped. Mills built the road for the silver mines of Candaleria and Cerro Gorda, and when they were worked out no others of equal importance were developed. He tried running the road for several years and then sold it to the Southern Pacific. Its length was 301 miles, and it is said that when he had finished signing the papers to the deal he remarked that he had built the thing 301 miles too long and 301 years too soon.

And there are several other somewhat similar cases that could be mentioned; some have their grim humors of unforeseen vicissitudes, and some are simply huge engineering blunders. the want of foresight and the want of system of planning for the future.

In general the principle of locating a cheap "grass roots" road is not bad, provided it is so located that it can be afterwards reduced in grade without departing materially from the right of way. But this was not done, and in many cases it is not being done today. The managements have not appreciated the importance of having the conditions graphically depicted to them, but now both managements and engineers are fast coming to their senses, and it is hoped that this discussion will tend toward that end.

Case after case of blunders can be cited. Take, for instance, the Wisconsin, Minnesota & Pacific. Towns have grown up on it that cannot be abandoned. If it is remodeled, two lines will have to be maintained. And there is the Tacoma-Tenino cutoff on the Northern Pacific in Washington. A 1.6 per cent grade was considered suitable when the original line was built, and it probably served the purpose for years, but at the same time there was a better route that could have afterwards been revised, and it was not taken. Towns are now built on the old line, and they cannot be abandoned, so the N. P. will have to maintain both lines.

Probably every one in the railroad engineering business in the United States knows of the Sherman Hill monument in Wyoming. It is at the summit of the great divide of the continent, and is supposed to commemorate the completion of the first trans-continental railroad—the Union Pacific. It is a huge affair, and probably cost some $20,000. It was scarcely completed before it was found that a grade revision would be necessary, and the new alignment now crosses the divide several miles to one side of the monument. So there it stands alone; no passengers ever see it; sheep herders alone visit it, and if it is a monument to anything, it is to engineering and administration blunders—an eloquent argument in favor of extensive topographical maps and plaster casts.

One reason railroad engineers may be opposed to extensive topographical maps is, that they have not been out with the U. S. Geological Survey parties, and have no idea how rapidly that organization works up a country, and have not realized now well some of those methods could be applied to railroad work.

In regard to "feeling the country out" with preliminary lines, samples of blunders from this method are legion. Take the Great Northern in the Flathead Valley in Montana. They thought they "felt" their way out, but they put their foot in. Of course they were in a hurry at the time. But later the best engineering talent in the country was used to correct the blunder. Kalispell then found itself on a branch road, and many miles of line were abandoned. It was a wooded country, and at that day and age "feeling the way" was the only method known. But what a difference it would have made if they spent more money to investigate, and taken the other route to start with. It would take many pages to recite all these blunders, but only a few of them are due to the engineers; the majority are due to an inexperienced administration of affairs technical.

In this comment theorizing has been avoided, and only facts cited that must carry their own lesson. For personal information Mr. Butler is cited to his own neighborhood. Mr. George Kopp, the originator of traffic diagrams, is Division Engineer on the Western Pacific at Oroville, Cal. Mr. J. Q. Barlow, Assistant Chief Engineer Southern Pacific R. R., is familiar with the J. D. Isaacs diagrams. His office is in the Flood Building in San Francisco. Very truly,

H. H. Edgerton, Jr.

463 Baltimore & Ohio Depot, Chicago, Ill.
April 3, 1914.

Sewerage

Operation of the Sewage Disposal Works at Atlanta, Ga.

Considerable publicity has been given to the design of the Atlanta sewage disposal works. The impression probably exists in many quarters that the works have now been in operation for some time. This is only partially true. The smallest plant was placed in operation in August, 1912, the largest in October, 1913, and the third plant will not be ready for service until about the first of May. In the operation of the completed works some interesting points have been observed and are here discussed. We are indebted to a paper before the American Association for the Advancement of Science by Capt. R. M. Clayton, superintendent of construction, and Mr. W. A. Hansell, Jr., assistant superintendent of construction, of the city of Atlanta, for the information here given

About 1888 Capt. Clayton as City Engineer, called Dr. Rudolph Hering in consultation for the design of a complete sewerage system for Atlanta. The design was for a combined system discharging into the water courses down which the sewers were constructed. Atlanta is on a ridge dividing the Gulf of Mexico from the Atlantic Ocean and the water courses headed from springs in the city At the outflow of the sewers there was no stream except the sewage and the water courses became a nuisance Numerous complaints began to come in from property owners along the streams and the city appreciated the fact that some remedy must be found for this condition In 1909 Dr Hering of Hering & Fuller was again called in consultation with Capt Clayton and a disposal system was designed

There are three main drainage areas in the city taking care of most of the sewage and it was decided to build plants for each of these territories carrying the sewage to them by gravity and to pump the sewage from the smaller areas into the sewers of the most convenient large area.

Interceptors are provided at the outfall of the main sewers. These interceptors are designed to carry eight times the normal flow of sewage and at the plant a by-pass is provided for the amount of sewage not treated. The plants are supposed to treat twice the normal flow and by-pass the balance but the tank capacity is found sufficient to treat more than this and the additional amount is by-passed between tank and filter as there is no excess filter capacity.

As these plants were among the first in this country to use Imhoff tanks the city officials were anxious to try out one plant and then, using the experience gained, proceed with another, but the property holders affected would not consent to the delay on the other plants and all three contracts were signed during 1910. The smallest plant, Proctor Creek, treating 3,000,000 gals. per day, was completed and put into operation in August, 1912. The largest plant, Peachtree Creek, treating 8,000,000 gals. per day, has been in operation since the middle of October, 1913. The third plant, Intrenchment Creek, treating 5,000,000 gals., will be ready about May 1, 1914. The excessive time for the construction of these plants was caused by the inability of the contractor to secure stone of the proper size for the sprinkling filters.

The designs for the three plants are practically the same and the methods of construction very similar. The idea in general is to screen out sticks, cans, dead animals, etc., by a fixed screen, then settle the sand in grit chambers, next take out all remaining settleable solids in the Imhoff tanks and let the bacteria in the sprinkling filters complete the treatment. No effort is made to make drinking water of the sewage and the reduction of the bacteria, *per se*, is not one of the aims of the plant but is accomplished incidentally The intention of the treatment is to secure a non-putrescible effluent

When the first plant was started it was found that more sand came down to the plant than was contemplated and a hopper shaped pit with a flushing valve at the bottom has been added where the coarser sand is settled before the sewage starts through the plant proper During a storm or at such other time as found desirable this valve is opened and accumulated sand is blown out into the stream. The valve is sometimes left cracked just enough to keep the sand moving. It is found that this pit more than pays the cost of installation each year in labor cost saved in handling the sand out of the grit chamber.

Leaving the sand pit the sewage passes through the screen which is of inclined iron bars with spaces of about $1\frac{1}{2}$ in opening. This screen is cleaned off whenever the accumulation is enough to retard the flow appreciably, and the screenings are burned or buried.

Passing the screen the sewage goes to the grit chamber which consists of a series of channels of such sectional area as to reduce the velocity to about 1 ft. per second. This retarding of velocity causes the settling of nearly all the remaining sand but not that of organic matter. During storm flow a small amount of very fine sand is sometimes carried beyond the grit chamber on account of the increased velocity, but this amount is so small as not to cause practical inconvenience. The grit chamber channels are three in number and so arranged that one or all can be used at a time, thus giving opportunity for cleaning out the sand deposited in one channel while operating the plant.

From the grit chamber the sewage goes to the channel of the Imhoff tanks There is a channel on each end of the Imhoff tanks with [illegible] over so that either one may be influent or effluent channel The sectional area of this channel must be such that there will be no decrease in velocity from the grit chamber or else there is settlement of organic matter in this channel The sewage passes over a wier from this channel into the Imhoff tanks, through the tanks over a corresponding wier and into the effluent channel. The Imhoff tanks consist of a number of series of three units each with circular decomposition chamber 25 ft. in diameter, with bottom of baffle slabs 11 ft 3 ins from the bottom of the tanks The top of the water line is 13 ft. 6 ins. above the bottom of the baffle slabs, and the slabs are continuous from inlet wier to outlet wier, thus giving a rectangular flow chamber over three disconnected decomposition chambers. Each decomposition chamber is provided with side and center vents for the escape of the gases of decomposition. The sewage was expected to take three hours for its passage through the tanks. This time is now considered too long, and has been reduced already by an increased amount of sewage; and as the city grows and the amount of sewage is further increased, the shorter time of settlement now advised—namely, about two hours—will be obtained. The direction of the flow of the sewage across the Imhoff tanks is reversed about every two weeks and in this way the amount of material in the decomposition chamber is kept as nearly uniform as may be. Unless the flow were reversed nearly all the solids would settle in the first unit and it would fill so rapidly that there would not be sufficient time for decomposition in this unit. The time for decomposition is a very variable one, depending on what materal comes into the sewer in the trade wastes, and no fixed time can be given, but it averages about three months. The sludge is taken out of the bottom of the tanks by means of a cast-iron pipe fitted with a valve and discharging into a sludge channel under a 6-ft. head of sewage. In order to avoid the formation of a solid mat in this sludge chamber lead pipes are provided, connected with the water supply system, and the deposited solids are stirred up about twice a week. The amount of sludge drawn out is determined by the condition of the sludge as it reaches the sludge channel, the valve being immediately closed on the appearance of undecomposed sludge. As discharged the sludge has from 85 to 90 per cent of water when turned onto the drying bed, and requires from five days to several weeks to become spadable. In warm dry weather it dries very rapidly, but in cold damp weather a much longer time is required. The amount of sludge accumulated in a month is very variable as the suspended solids in the sewage vary from about 100 to nearly 600 p. p. m., and have averaged about 250 p. p. m. The suspended matter in the effluent from the tanks does not vary anything like that much, going from about 70 to 100 p. p. m. and averaging 85 p. p. m. The settlement in the tanks is practically perfect and the operators were unable to make a test of the tanks using baffle boards as compared with the tanks with the boards removed, because there were no settleable solids left in either case.

It was anticipated that there would be some colloidal matter, hair, etc., which would pass through the tanks, and a screen of fine-mesh bronze wire was provided at the Proctor Creek and roughing filters at the Peachtree Creek plant. There is nothing in the tank effluent for either method to catch.

The Imhoff tank effluent goes to the dosing tanks, which are measuring tanks to measure the dose given the filters. Twenty-four-inch automatic siphons regulate the head at which the filter nozzles start and stop, and provide intermittent dosing of the filter stone. As set at present it takes about twice as long to fill up as to discharge the dosing tank, and the cycle at Proctor Creek is about 12 minutes, while at Peachtree Creek it is about 20 minutes The sewage is distributed to the nozles in the filter by large pipes feeding 6-in. distributor pipes with the nozzles attached to the distributors. The filter stone is 6 ft. deep, varying in size from 1½ to 2½ ins., and rests on split terra cotta half-pipe imbedded in a concrete base arranged in inclined planes and discharging into channels which in turn discharge into an effluent pipe with a trapped manhole emptying into the creek.

A special feature of the sprinkling filters is the arrangement for furnishing additional oxygen to the bottom of the broken stone. This consists of a cowl with vane attached, causing the opening always to face the wind. The air blown down is distributed through the split pipe and rises through the broken stone. It is expected that tests will show this arrangement to be of considerable value, but so far the operators have not been able to give proper tests.

The nozzles used are of the Miller and Weand type, set 13½ ft. centers, staggered, and giving circular sprays. The angle of discharge into the air is about 60°, and gives a high travel through the air. An interesting thing about this long air travel of the spray is that the sewage during its passage through the air absorbs about 60 per cent of the total oxygen absorbed after leaving the dosing tank. This absorption suggests some very interesting experiments as to methods of avoiding the use of the present expensive sprinkling filter.

When the sewage reaches the plants it is dark colored, fairly fresh and moderately offensive to the nose and eye. When it leaves the Imhoff tank it is nearly clear and only slightly odorous, and when discharged from the sprinkling filter it is as clear as the country streams and as odorless as the water from the "old oaken bucket."

A chemist, bacteriologist and an assistant test the operation of the plants daily, and day laborers keep the plants clean. Night watchmen are on duty to be at hand in case anything goes wrong during the night. It is absolutely necessary to give the plants competent daily attention as they will not take care of themselves any more than other expensive machinery, and it is not fair to the method of disposal or to the parties furnishing the money for the construction to turn sewage into an Imhoff tank and leave it to work out its own slavation.

One great trouble with all the screens is that the stream beds are never cleared of trees and other debris which form obstructions and cause overflows of the adjacent land. This year the city cleared about two miles of the Peachtree Creek bed, and changed it from a sluggish, muddy-bottom stream to a sandy, sparkling one with the velocity so increased that overflows are reduced to a minimum. By this work and the construction of the plants the streams have been rid of unsightly places offensive to the eye and nose. These streams have once more become cleanly country branches and the kicking property holders along them have changed to friendly neighbors. These changes have been wrought mainly by the Imhoff tank.

Suggestions on Designing Sewage Treatment Works for Saskatchewan Cities.

A recent bulletin prepared by the provincial Bureau of Public Health contains suggestions on designing sewage treatment works for municipalities in the province of Saskatchewan. The suggestions which are offered as a guide to designers, rather than as definite rules to be observed under all conditions, are here reprinted as reported in a recent issue of the Contract Record:

Screening.—(a) That where sewage is to be pumped, provision be made for screening, cleaning the screens at regular intervals, and for removing and disposing of screenings.

(b) That where works are of a character producing a large quantity of screenings, mechanical means be provided for their removal.

Grit Chambers.—(a) The function of grit chambers (or detritus tanks) being to arrest the heavier mineral particles carried in suspension, the construction of such tanks is, generally speaking, unnecessary in this province, where the majority of sewage systems are designed on the "separate" principle.

(b) That in special cases where sand or other mineral particles cannot be kept out of the sewerage system, it is advisable to introduce grit chambers.

(c) That these be constructed in multiple compartments to take care of the varying volume of flow.

(d) That a lineal velocity of 1 ft. per second be aimed at, calculated to retain the heavy mineral, but not the organic matters.

Pumping.—(a) That where it is necessary to raise the sewage at the works, it is advisable that all machinery be in duplicate with alternative forms of power in case of failure.

(b) That appliances for raising sewage be specified with reference to efficiency in dealing with solids.

Sedimentation.—(a) That there be at least two sedimentation tanks.

(b) That such tanks be so constructed that the precipitated solids are automatically and continuously removed from that portion of the tank in which precipitation takes place, and that a tank or chamber combined with, or separate from, the sedimentation tank, be provided, into which the precipitated solids may pass by gravitation immediately following treatment.

(c) That consequent upon the modern requirement of the continuous removal of sludge as above stated, all base slopes of sedimentation tanks be made as near to the perpendicular as is practicable, relative to general construction.

(d) That the tank capacity be equal to one-fifth of the dry weather flow in 24 hours, or equal to 3 hours' flow calculated upon the 24 hours' dry weather flow taking place in 15 hours.

(e) That the cross sectional area of the tanks provide a velocity of flow of not more than 0.05 ft. per second, while lower velocities are preferable. Flows may be either vertical or horizontal.

(f) That consideration be given to the design of the inlets and outlets with a view to ensuring uniformity of flow throughout the breadth of the tank, and the absence of stagnant sections; and that all channels and parts of the tanks apart from the sludge storage area, be so constructed that no solids are retained.

Sludge Storage.—(a) That the overall depth of the sludge storage chamber from the surface of sewage in sedimentation tank be generally not less than 15 ft. Greater depths may be adopted, producing a more concentrated form of sludge.

(b) That in deep tanks, wherever possible, provision be made for breaking up the sludge at the inlet to the sludge removal pipe.

(c) That the capacity of the sludge storage chamber be equal to at least four months' precipitation of sludge, containing 85 per cent of water. Greater storage capacity is preferable as septic action is delayed in winter months. The cubic capacity of the sludge storage chamber shall be taken as only that space which is below the level of the deepest point of the sedimentation tank. In general, the average accumulation of sludge may be taken as 3½ cu. yds. per 1,000,000 gals. of sewage on the above basis of dilution.

(d) That ample provision be made for the escape of gases from the surface of the sludge storage chamber.

(e) That pipes for the conveyance of sludge be of an internal diameter of not less than 8 ins. and that the inclination of such pipes, where the sludge is discharged by gravity, be at least 3 per cent and preferably 5 per cent.

Biological Filtration.—(a) Where a dosing or siphon chamber is constructed to regulate the flow of the sewage over the surface of filter beds, that the capacity of such chamber does not exceed a ratio of 2 gals. of sewage to each square yard of filter surface. For instance, if the area of the filtering surface be 200 sq. yds. the capacity of the dosing chamber should not exceed 400 gals., representing a dose of ½ in. depth of sewage over the whole surface of the filter.

(b) That the depth of the filter media be not less than 4 ft. and preferably 7 ft.

(c) That the filter media for effluents from the above form of tank be composed of hard broken stone or other suitable material, broken from 1 in. to 2 in. cubes.

(d) That the surface area of filtering media for domestic sewage be in proportion to the population using the sewers, i. e., in proportion to the amount of oxidizable matter present in the sewage.

Where a high degree of oxidation is required, the ratio of population to surface area of filter media should be approximately 17,500 persons to the acre (or 275 sq. yds. per 1,000 population).

This corresponds to a rate of filtration of 1,750,000 imp. gals. per acre per day, or 155 imp. gals. per cubic yard per day (assuming depth of filter media to be 7 ft. at a per capita flow of 100 imp. gals. per day.

The efficiency of filters will not be materially affected by increasing the rate of filtration during periods of storm to say three times the above stated rate, provided that the increase in volume is due to clear water.

In cases where the volume of dilution at the point of final discharge exceeds 20 times the volume of the sewage effluent, higher rates of filtration may be adopted.

(e) That the method or apparatus adopted for the distribution of liquid over the filter bed shall ensure a uniform distribution over the whole surface.

(f) That all filter beds be drained at the base by the pipes or preferably by means of a false floor over the entire base of the filter.

(g) That sufficient provision be made for ventilation to allow of oxygen being present at all times and in all parts of the filter bed.

Humus Settling Tanks.—(a) That it is generally advisable to provide settling tanks for the removal of the humus which is unloaded by the filter beds from time to time.

(b) That such tanks be constructed in every case where disinfection of the final effluent is adopted.

(c) That humus tanks have a capacity equal to 1-45 of the dry weather flow in 24 hours, or equal to 20 minutes' flow calculated upon the 24 hours' dry weather flow taking place in 15 hours.

(d) That it is desirable that such tanks have separate storage compartments as in the sedimentation tanks above described.

Disinfection.—(a) That where chloride of lime is used for the disinfection of the final effluent, provision be made for weighing, measuring, and storing the disinfectant in a dry covered building.

(b) That the period of contact between the disinfectant and the sewage be not less than 15 minutes. An open pond or lagoon will serve this purpose.

Effluent.—That the effluent from the works be discharged into the watercourse or lake in such a manner as will ensure a maximum amount of dispersion.

Housing of Tanks and Beds. (a) That all parts of the works containing sewage under treatment be housed to conserve the latent heat of the sewage during low temperatures and to prevent fly nuisance in summer.

(b) That such covers be designed to enclose the minimum amount of space, at the same time giving room for accessibility and inspection.

(c) That with properly designed covers the introduction of artificial heat is unnecessary in sedimentation tanks, and may be obviated in filter beds.

(d) That provision be made in all covers for the access of light and for efficient ventilation.

Laboratory. That wherever the size of a municipality or other circumstance warrants its construction, a small laboratory be provided and equipped in which simple tests may be made of the sewage and effluents.

Laying out the Surrounding Grounds. That provision be made in the specifications and estimates for laying out, grading and improving the appearance of the surrounding grounds by terracing and seeding the slopes.

On Choosing Sewage Treatment Processes Suitable to Local Conditions, with General Data on Costs.

Few problems that come before the sanitary engineer are more difficult of exact solution than the determination of what manner of sewage treatment to employ in order fully to meet the requirements of a local situation, and at the same time to give just and adequate consideration to interests farther down the stream. It is a problem demanding for its best solution sound and mature judgment as to what is practicable and fair and in accord with good public policy, no less than thorough-going knowledge of the physical facts involved. There can hardly be too much repetition of the principles underlying the proper choosing of treatment processes suitable to general types of local conditions. With this thought in mind we are here quoting matter bearing upon this problem from a paper read before the Michigan Engineering Society by William C. Hoad, professor of sanitary engineering, University of Michigan.

No definite rules can be laid down as a guide in the many and varied situations that arise, but in general it may be said that there are two main cases, namely: the case in which the chief purpose of the sewage purification process is the protection of public water supplies from unnecessary and unreasonable pollution, and the case in which the principal object to be attained is the avoidance of offensive conditions in and along the watercourse below the point of discharge.

In the case first named the most important thing to be accomplished is the removal or destruction of the sewage bacteria, these including, of course, whatever disease-producing forms may be present. This may ordinarily be accomplished by a suitable filtration process or by sterilization, or by combination of these processes. But to be economical, filtration should be preceded by some clarification process, such as sedimentation or treatment in septic tanks or in two-story tanks; and to insure that any sterilization process may be effective and reliable it is necessary that the sewage to be sterilized should first be thoroughly clarified.

In the case involving only the avoidance of offensive conditions in the stream below the sewer outlet the requirements may range all the way from a mere screening of the sewage through moderately fine mesh screens to an elaborate treatment scheme involving detritus removal, clarification, sludge disposal, filtration, and the final sterilization of the effluent. The separation of the sludge alone may be sufficient in case the only thing to be avoided is the formation of sludge deposits in or along the stream below the point of discharge, such as in a pool in the stream or in a mill pond, or along the shore of a lake. In the case of discharge into a very small watercourse, in which a minimum flow of water is not sufficient to assimilate the sewage without offense and a nuisance results, a sludge removal process alone will not ordinarily be sufficient, and must be supplemented by filtration for the oxidation of the unstable organic matter contained in the sewage. This is a case of frequent occurrence.

Obviously no very satisfactory figures can be given for the cost of sewage treatment, as the expense will depend largely upon local conditions. In a general way, however, it may be stated that in the case of a city having an efficient system of sanitary sewers from which all storm water and practically all ground water are excluded, the first cost of a treatment plant consisting of septic tanks or two-story tanks for the preliminary process and broken stone or gravel or cinder contact filters for the finishing process may be taken at about $10,000 for each 1,000,000 gals. daily capacity, or about $1 per capita for the population served by the plant. This does not include any intercepting sewer, pumping plant, or long outfall sewer that may be necessary. The total yearly cost, including expenses of operation and maintenance, interest on first cost, and depreciation, may be taken roughly at from $12 to $15 per 1,000,000 gals. treated, or about $0.50 per year per capita of tributary population. If it is necessary to pump the sewage in order to purify it, this service will usually increase the per capita cost to $0.75 or $1 per year.

These figures for the cost of sewage disposal are much higher than those frequently advanced. In fact communities oftentimes have been led, or permitted, to believe that the first cost of a sewage treatment plant is its only cost unless the sewage has to be pumped, in which case the cost of running the pumps must be considered. It is true that the amounts spent in routine operation are very low in the case of many disposal plants operating wholly by gravity and controlled by automatic regulating devices, particularly during the first few years following their installation when their loads are light. But even under such favoring conditions if all the items of cost be taken into consideration—the operating expense, the interest on capital invested, and the depreciation—the total annual cost for each person tributary will not often fall much below the figure mentioned.

The value to a city of the use of a flowing stream of water for the purpose of disposing of its sewage by dilution may be approximately determined from the cost figures given above. Assuming that the sewage will be quickly and inoffensively assimilated if the stream flow amounts to 5 cu. ft. per second for each 1,000 of the sewered population, it follows that each cubic foot per second of the minimum discharge of the stream may be taken as being worth approximately $100 per year for this purpose. This is the value if it is possible to purify the sewage in an artificial treatment plant without pumping and without the construction of long and costly intercepting sewers and outfall mains. This figure would be increased from 50 to 100 per cent if the operation of the disposal plant required the sewage to be pumped, and would be driven still higher if other costly items of construction were necessary. All this, of course, is upon the assumption that the contamination of no public water supply is involved and that the disposal of the sewage of the city by suitable dilution in the stream is permissible.

Desirability of Sewage Treatment Experiments at Universities—Specific Suggestions for Proper Operation of Municipal Plants.

Although a great deal of experimental work has been done by the various universities on testing materials of different classes, but very little experimental work has been done by such institutions on sewage disposal. We have urged the desirability of such experimental work and have recently published an article setting forth the object and design of such an experiment station at the University of Michigan. In a committee report on sewage disposal to the Indiana Sanitary and Water Supply Association Mr. Charles Brossmann, consulting engineer, Indianapolis, Ind., also emphasized the need of such experimental work. Mr. Brossmann also made specific suggestions for the proper operation of different types of municipal sewage disposal plants. The portion of the report bearing on these points follows:

SEWAGE DISPOSAL EXPERIMENTS AT UNIVERSITIES.

It is no doubt true that for the larger cities it is necessary to conduct special investigations and experiments because of the complexity of city sewage, due to various industrial wastes and other factors which do not always allow of an immediate solution. Still in smaller plants that are used purely for sanitary disposal and for certain localities the modern university might do much toward standardizing designs or methods to be followed in a general way for the smaller places. Many places are found to have conditions which are practically the same, and such experiments would be of value. There is a good field for research work on a small scale, which at the same time would be of great educational value to the students engaged in the investigations.

There is still much to be done in reducing to more definite standards the question of flows, action of sludge, size of settling and digestion chambers, periods of retention, and various other questions in connection with Imhoff tanks. There is room for improvement, and consequently need for experiment on a number of questions of detail relative to such tanks, and to various phases of filtration treatment such as underdrainage and retention. Such experimental plants operated either by the state or by institutions would prove of great value if they were properly conducted.

SUGGESTIONS ON MUNICIPAL PLANT OPERATION.

Persons in charge of sewage disposal plants should understand the principles of operating them, and should be thoroughly familiar with plant details. Operators should understand the dosing apparatus and know how to adjust same in case it gets out of order. They should be able to tell when the sedimentation or reduction tank is operating properly; whether too much suspended matter is being carried over to the filters, or whether too much sludge is in the tank. Also they should know when contact beds are being overworked, or when sand filters are clogged or need to be raked. It does not take a technical man to know these things, but it requires a man who will look after the plant at intervals and keep track of what it is doing. At municipal or public plants the city or county engineer could direct such work, and at public institutions the chief engineer at the plant is fully capable of getting results if he will devote the time necessary for that purpose.

Too little attention is usually given to the operation of sewage disposal plants, and the plants are often left in charge of the poorest kind of labor. It is quite evident that instructions should be given to the operators of such plants if proper results are to be obtained and the aims of sewage disposal are to be accomplished.

The following suggestions are given as applicable to the proper operation of the smaller plants which cannot receive expert supervision.

Plain Septic and Imhoff Tanks.—As the purpose of such tanks is mainly for the separation and retention of the suspended matter to as great a degree as possible, the tanks should be kept in such condition that this will be accomplished. Otherwise the efficiency of such tanks will be impaired.

To get best results large accumulations of septic sludge should be avoided when they cause violent ebbulition and a consequent carrying over of the sludge to the filters. The sludge should be removed when necessary.

In using Imhoff tanks the sludge should not be allowed to get above the depositing slots at the bottom of the settling chamber.

Large and unusual flows through the tank, such as storm water flows, should be diverted, especially in plain septic or sedimentation tanks. The removal of suspended matter should be as complete as possible, as the less this matter reaches the filter beds the less trouble there will be in operation. Sludge should be blown off when possible at regular intervals, or at time of high water, unless special provision is made for sludge drying.

Contact Filters.—The main trouble with contact beds is the tendency toward clogging. Contact beds should not be overworked and should have stated periods of rest. It is impossible not to get some matter carried over from the tank, but this can be taken care of in a way to cause the least trouble, and to catch most of the material before it works into the beds. Proper and complete underdrainage will assist greatly in getting rid of accumulations, but it is safe to say that few contact beds are very thoroughly underdrained. Too little regard is usually given to good underdrainage and proper bottom construction to contact filters.

Sand Filters.—The sand filter is not so liable to trouble as the contact bed, but it goes without saying it should be kept as free as possible from material that will clog. When ponding occurs the bed should be raked about ¾ in. deep. The bed can be allowed to dry and the top skin will peel easily and can then be raked off. Slopes around filters should be sodded so that clay or dirt will not get into the sand and clog it.

Low spots on the sand area should be kept filled so that the sewage will flow level and all parts worked as uniformly as possible, and in freezing weather the surface of the filter should be furrowed.

Roads and Streets

Methods and Costs of Constructing Gravel and Sand-Clay Roads in Perry County, Alabama.

Contributed by George C. Scales, Senior Highway Engineer, U. S. Office of Public Roads, Washington, D. C.

In November, 1911, Perry County, Alabama, voted a bond issue of $110,000 for good roads, which with the premium and accrued interest on bonds, bridge fund and state aid amounted to over $122,000 available for road and bridge construction.

In order to harmonize many conflicting elements and satisfy opposition from the country districts, it was decided that $45,000 should be eually divided among the five roads radiating from Marion, the county seat; $20,000 for the roads in the outlying districts and the remainder $57,000 upon the road leading from Uniontown, the principal city in the southern part of the county, to Marion.

LOCATION OF ROADS CONSTRUCTED.

There are three roads leading from Uniontown to Marion and in May, 1912, the writer was employed as engineer and directed to report on all the roads, the cheapest and shortest route to be selected by the Commissioners. It was shown that the longest route would cost $100,000, and that either of the other two would cost about $85,000. The "Lower Road," 17.63 miles between corporate limits, was finally selected and as there were not sufficient funds to build the entire road, it was ordered that work be conducted from each end of the road and the same amount should be expended on each end.

Soil Conditions.—Uniontown is in the "Black Belt" of Alabama and has the heavy black alluvial soil which when thoroughly wet becomes sticky and slippery and almost impassible, while the soil in and about Marion is mostly a good quality of red clay with occasional stretches of sand, pipeclay and sandstone.

Traffic.—Like all towns in Alabama, the principal traffic consists of farm wagons with tires of 3 ins. or less width carrying loads of cotton or fertilizer weighing 1,000 to 2,000 lbs., exclusive of the wagon. The traffic is in one direction, going to town in the morning and returning at night, which makes a narrow, hard surface more permissible than if sufficient width had to be provided for the frequent passing of vehicles.

GRAVEL ROADS

Selection and Cost of Gravel.—It was decided to use gravel on the black land, but the only gravel in the county is at Highberger, 36 miles from Uniontown and 17 miles from the railroad at Marion, or on the banks of the Cahaba River, 25 miles from Uniontown and 7 miles from Marion.

After some search the engineer found a bed of good gravel overlaid with red clay a short distance from the railroad at Potter Station in Dallas County, 23 miles from both Marion and Uniontown by rail.

A contractor was induced to lay one-half mile of track to the pit and put in a trap to load the gravel on condition that the county would agree to pay the freight and take not less that 9,000 cu. yds., paying 40 cts. per cubic yard for gravel properly mixed on the cars.

The engineer secured a special freight rate of 25 cts. per ton from the Southern Railway Co. for all gravel to be used at Marion and Uniontown on road improvement. This made the cost 78 cts. at these cities instead of $1.20 per cubic yard, which was the lowest price to be obtained from the nearest pits of good gravel in Dallas and Montgomery Counties.

Physical Characteristics.—Five miles of road at the Uniontown end were first surveyed and advertised for bids, the maximum grade being 4 per cent; width of graded road 20 ft. between ditches; and gravel surface 12 ft. wide and 9 ins. thick before compression. The specifications called for grading with 500 ft. free haul and a price per mile per cubic yard for hauling gravel.

D. R. Cook of Selma, Ala., was the lowest bidder at the following prices:

Earth grading, per cu. yd.	$0.25
Loose rock, per cu. yd.	0.50
Solid rock, per cu. yd.	0.90
Overhaul, per cu. yd. per 100 ft.	0.01½
Clearing and grubbing, per acre	75.00
Subgrading, per mile	150.00
Hauling, per cu. yd. per mile	0.40
Gravel loaded, spread, rolled and maintained 30 days, per cu. yd.	0.15

Method of Compacting.—Soon after starting graveling it was decided that rolling could be dispensed with as the teams were hauling 1½ cu. yds. of damp gravel, making the total weight of wagon and load about 5,000 lbs. on tires 4 ins. wide, which give sufficient compression; a road machine was run over the gravel frequently and a perfect surface was easily obtained.

All intersecting roads and lanes were given a coating of gravel from 50 to 100 ft. away from the road to prevent mud being brought on to the improved road.

Cost.—The total cost of the 4.6 miles of road built was $23,462.84 or $5,100.63 per mile, distributed as follows:

17,282 cu. yds. graded at 25 cts.	$ 4,320.58
3 acres cleared and grubbed, at $75	225.00
1½ miles subgraded, at $150	675.00
8,176 cu. yds. gravel loaded, spread and maintained, at 15 cts.	1,226.34
607.5 cu. yds. gravel hauled 1 mile, at 40 cts.	243.00
1,782 cu. yds. gravel hauled 2 miles, at 80 cts.	1,425.60
2,961.0 cu. yds. gravel hauled 3 miles, at $1.20	3,553.20
2,824.0 cu. yds. gravel hauled 4 miles, at $1.60	4,518.40
8,830 cu. yds. gravel loaded at pit at 40 cts.	3,532.00
8,830 cu. yds. gravel, freight at 25 cts. per ton	2,958.36
Engineering, inspection, etc.	785.36
Total	$23,462.84

Culverts and Bridges.—Culverts up to 36 ins. diameter were of corrugated ingot iron pipe in half sections. Bridges were of the I-beam type with concrete floor up to 25-ft. span and steel truss bridges with concrete floor were used for all spans over 25 ft.

All bridges were designed for 16-ft. roadway with 6 ins. gravel fill and a live load of 15-ton traction engine. They were built by county forces at a cost of $12 per cubic yard for the concrete. Labor and material prices were as follows:

Foreman, per day of 10 hours	$ 4.00
Labor, per day of 10 hours	1.50
Team and driver, per day of 10 hours	4.00
Lumber, per M ft.	20.00
Cement, per bbl. in sacks	2.40
Sand, per cu. yd. f. o. b. cars	1.00
Screened gravel, per cu. yd. f. o. b. cars	1.15

Of the 7.6 miles of road built at the Marion end of the Uniontown road, 4 miles were through black land and the gravel at this end of the road was hauled from the railroad. A dead haul of 3.2 miles, and from that up to 8 miles, so the cost was very high.

Special Method of Construction, Using a Small Amount of Gravel.—As there were only 3,000 cu. yds. of gravel available for this distance, the experiment was tried of shaping up the road with a crown of 1½ ins. to the foot, or 15 ins. for a 20-ft. roadway, and then spreading the gravel in two strips 6 ins. deep and about 3 ft. wide equally distant from the center of the road, to cover the ordinary wheel tracks. After each rain this was shaped up with the road machine and kept dragged until dry and hard and more gravel added where soft spots developed. In a few weeks the road was in good condition with a graveled surface about 8 ft. wide, with only one-third the usual amount of gravel used per mile.

SAND CLAY ROADS.

The Commissioners were dissatisfied with the bids received for the sand clay roads leading out of Marion, so all bids were rejected and they decided to have a mile of sand clay road built under the direction of the engineer to see how the cost would compare with prices bid by contractors. A mile having three heavy hills with grades up to 20 per cent was selected for the experiment. The county owned a large road machine and purchased 8 drag scrapers, 6 wheel scrapers, 2 road plows, shovels, mattocks, and all tools except the road machine were charged directly to the job.

Labor prices were as follows:

Foreman, $3 per 10-hour day; labor, $1 per 10-hour day; team and driver, $3 per 10-hour day; loader, dumper and plowman, $1.50 each per 10-hour day.

Dynamite at 18 cts. per pound was used freely on the hard clay banks and this mile was graded at a cost of $2,700 or 22½ cts. per cubic yard, or 2½ cts. less per yard than the lowest price bid. Grades were reduced to 6 per cent and 8 per cent and this was so satisfactory to the Commissioners and general public that the engineer was ordered to build all the roads radiating from Marion in a similar manner.

All equipment purchased was charged to the road on which it was used and 22.5 miles were accordingly built at an average cost of $1,615 per mile. The grading was very heavy on four of the five roads built, but some miles were constructed for about $800. In the outlying districts 37.4 miles of roads were ditched, graded to a width of 18 ft. between ditches and properly crowned at an average cost of $435 per mile.

The Marion end of the Uniontown road passed through 3.2 miles of sand-clay and the rest of the way black land, and as there was not sufficient money to improve the entire road between the two towns, it was desired to have the improved gap as small as possible.

The first 1.9 miles of sand clay were built by county forces and the other 1.3 miles by contract at 20 cts. per cubic yard for grading with 500-ft. free haul and 1 ct. per cubic yard for overhaul.

This section included a fill 1,600 ft. long on the town side of Bogue Chitto Creek bottom that averaged 4 ft. high and contained over 6,000 cu. yds., which was made from a bank on the further side of the creek and by so doing shortened and straightened the road and reduced a heavy grade to 4 per cent.

Clay was laid in place on this section with 500 ft. average haul for 35 cts. per cubic yard and mixing with sand cost 1½ cts. per square yard. The mixing consisted of plowing, disc harrowing and shaping with a road machine for 30 days.

PROVISIONS FOR MAINTENANCE.

On the completion of the work a small road machine and a steel road drag were assigned to each of the seven improved roads and contracts were made with a man living on or near each road, whereby he agreed to drag or grade the road whenever ruts were made by the traffic after the rains.

These men were allowed $2.50 per day for their services and $3 per day for each team of two mules and driver. After the engineer left the Commissioner of each district was to determine when such work was necessary and telephone or get word to the overseer the night before. Several gravel dumps were made along the gravel roads so that they could be easily repaired in the future.

Nearly every one in the county is now a convert to good roads and on many of the country roads the farmers gave teams and men to work under direction of the foreman. In one instance 25 teams and the necessary men gave their time for two weeks hauling gravel and clay, grading the road and mixing clay and gravel for three quarters of a mile through the river bottom road and making a fine graveled road where it had formerly been impassable during wet weather.

Record Change of Pumps.—The crew of the 29-in. pipe line dredge No. 48, working at Margarita Point on the Panama Canal, recently established a new record for time in changing main pumps, by removing the old and installing new pumps in 21 hours. This is shorter, by three hours, than any previously recorded time.

An Interlocking Concrete Curb for Brick Pavements in Pierce County, Washington.

(Staff Article.)

In preparing plans for resurfacing in Pierce County, Washington, 4½ miles of concrete road with brick, the interlocking concrete curb shown by the accompanying drawing, Fig. 1, and designed by County Engineer Mr. Roy Thompson was adopted. The drawing, with the extracts which follow from the specifications, indicates already the construction of the curb:

(1) Forms.—Two-inch planks of width of interlocking curb, as shown on the plan, shall be laid flat upon each edge of the concrete roadway. These planks shall be laid with the inside edge true to line to be given by the engineer. The planks shall be held in place by stakes.

(2) Sand Cushion—Upon the concrete base there will be laid a coat of clean, dry sand, averaging 1½ ins. thick after rolling. The sand shall not be coarser than ⅛ in. and shall be laid against the inner edge of the forms described in section 1 hereof, and shall be spread with a template having a steel-faced edge cut to the true cross-section of the finished roadway.

(3) Brick—No course of bricks shall start nearer than .... inches nor farther than .... inches from the outer edge of the concrete base, the remaining space to be filled by the Thompson Interlocking Curb, hereinafter described.

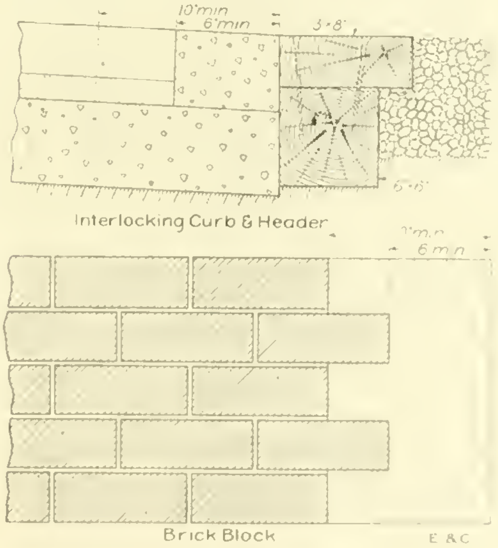


Fig. 1. Interlocking Concrete Curb for Brick Pavement.

(4) Wood Blocks—Wood blocks shall be used to complete each course of brick in place of bats; these blocks to be removed as hereinafter provided. * * *

(6) Interlocking Curbs—After the bricks have been rolled to the satisfaction of the engineer, the wood blocks and forms shall be removed, care being taken not to disturb the bricks or the sand cushion.

Forms not less than 2 ins. thick shall be stood on edge against the outside edge of the concrete roadway. The top of the form shall be true to the finished line and grade of the curb.

The Thompson Interlocking Curb is to be built, according to plan, with a 1:2:4 concrete. The mixture shall be made as provided under the head of concrete in the general specifications. The concrete shall be placed in the forms just ahead of the grouting crew, and in all cases not more than 30 mins. shall elapse between the time of placing the curb and the grout adjacent thereto. An expansion joint must be placed at the end of curb laid each day, and at such other points where it is found necessary to stop the grouting and laying of the curb for a period of more than 30 minutes.

The concrete is to be plastic when placed in the forms, and to be floated with a wood float immediately after the last application of grout. The outer edge of the curb shall be rounded to a 1 in. radius.

The curb as described has been donated by its inventor, Mr. Thompson, to the public.

The Value of "Road Days" as a Method of Road Improvement.

To THE EDITORS: Permit me to express my pleasure in your method of handling the volunteer road building question in your editorial on "Road Days and Road Data" in the issue of the 11th inst.

Your article embraces generally the sentiments which I expressed last September in an article printed in the Journal of the Engineers' Club of Baltimore, on "Road Building as a Pastime," which I enclose for your information and use should you desire.

Yours very truly,

WM. D. JANNEY,
Consulting Engineer.

Baltimore, Md., March 17, 1914.

(The following are important extracts from the article mentioned by our correspondent: Editors.)

The road days of Missouri and Arkansas were simply a return to obsolete system and amounted to no more than a pastime under the delusion that they were road building. The writer does not impugn the earnestness or zeal of those engaged in this picnic, nor does he assert that they did not do their best, but it appeals to him as a heedless and needless waste of vital force and money wealth, and instead of being heralded by the press the country over as a shining example of patriotism and state pride, it should have been commented on as a glaring example of the ordinary manner of conducting our public affairs.

These men were all, or nearly all, called from their usual avocations to perform, as stated, duties for which they were in no way fit. Each, therefore, lost absolutely his usual daily wage or income from his daily work. Now suppose instead of such a spectacle it had been decided that every man should pay into a road fund the equivalent of one day's earning at his regular work, his loss would have been only half, and if this fund had been spent under the direction of a trained man or commission the difference in results would be immeasurable.

In neither of these states was there at the time any idle labor, so that the lowest wage-earner would have contributed at least $1.50 to $2.00, and when we consider the sums paid by the professional and business men, the effect of the energy of these 500,000 days' work at their regular avocations would have provided nearly if not quite $2,000,000 available to pay efficient, regularly trained, physically able men to do perhaps three times the number of days' work of at least 90 per cent efficiency instead of the 60,000 or 70,000 days' work of actual efficiency value done by this crude working force of volunteers. What sad waste! What extravagance!

Of course an attempt at reply is made by declaring the impossibility, under our form of government, that this would be unequal taxation. Would it? Did each by paying in time alike only give to the state the same as all others? No! he gave, each the value of what he might have earned in the pursuit of his regular calling for the period worked.

The truth should be placed boldly before every individual, and the public at large educated more fully in the fact that, if time is money, then the best expenditure of time will bring in the greatest result in value, and any such indulgence as that discussed can be viewed in no other light than road building as a pastime, and in no sense as of real value to the community, but a dead loss to the individuals of it.

The writer thinks it a great mistake for the press to treat it otherwise, to the possible expansion of the idea and the detriment of earnest effort to improve our roads. It may perhaps be admitted that the experience gained possesses some value as demonstrating to those engaged the conditions of the roads and an added knowledge of what has to be done and what it requires to do it, and thus advance their education towards the point where they will more cheerfully submit to the proper assessments of money for this purpose. Should this result be attained it may prove of lasting benefit although at an exorbitant cost.

The matter of road building is a serious subject and, as the ultimate success depends so very largely on public sentiment back of it, it is possible such annual picnics may be the Chautauquas at which the education is gained to create this sentiment. The writer believes that our road engineers have not given the matter of education of the public that amount of attention it deserves. They have been so closely engaged studying the necessities, conditions, materials and methods that they have not given as great publicity to the results of good roads in their bearing to life, both working and social, of the communities, nor to show by comparisons the value in reducing costs of everything due to better roads.

Causes of Failure and Recommendations for Materials and Construction Features for Concrete Roads.—In a paper presented to the tenth annual convention of the American Road Builders' Association, Mr. F. F. Rogers, State Highway Commissioner, Lansing, Mich., states "that most of the defects in concrete roads can be traced to the following causes: (1) Too lean a mix; (2) A poor aggregate, (3) Poor foundation, including inadequate drainage; and (4) lack of suitable expansion joints properly spaced." He states "that a large percentage of the defects in city and small town concrete pavements may be charged to a lean mix, especially where two-coarse concrete has been laid." Under favorable conditions and with careful workmanship he believes "that the two-course pavement can be made a success. "The top layer must be put on before the bottom layer has had time to take the initial set, and the mix should not be leaner than 1:2½:5 for the base nor learner than 1:2:3 for the top. He prefers a one-course concrete with a 1:2:3½ mix, taking great care in the selection and grading of the materials forming the aggregate." In discussing aggregate materials he expressed the opinion "that the best grades of lime rock are worth considering in that connection, especially on the secondary roads. The lime stone (which is a dolomite) would at least wear evenly, which is not true of the cobbles used in Michigan, owing to the varying degrees of hardness and toughness in the different varieties of rocks found among the cobbles in our state."

Under the subject of expansion joints he stated "that experience has proved that from 25 to 33 ft. is a suitable spacing for transverse joints. By way of experiment a 150-ft. stretch of 16-ft. roadway was laid on Michigan Ave., Wayne County, this year, without expansion joints of any kind. Before the road was poned to traffic nature had supplied the transverse joint spaced from 18 to 26 ft., with one extra thrown in for good measure."

In concluding he remarked "that if all the knowledge we now have on this subject were utilized in design and construction, pavements and roadways of this material could be laid that would remain almost free from the defects named. There is some question, however, if to do so would not make the expense of the pavement or roadway so high as to be practically prohibitive."

Wood-Block Pavements in Berlin.—Wood blocks are used for paving the streets of Berlin only on the sharpest grades, to give a better footing, and on bridges and their approaches, to lessen the jar. According to a U. S. consular report, less than 2 per cent of the street pavement is of wood blocks.

The wood blocks most used in Berlin are Swedish pine and, to a more limited extent, the Australian hardwood varities—tallow-wood and blackbutt. The native pine and beech blocks are, however, also used in Germany, as are the American cypress and yellow and pitch pine. The wood used must be carefully selected, must be free from knots and cracks, and it is considered unwise to use in the same section of pavement woods from different countries, wood gathered at different altitudes, or wood of different species.

The dimensions of the paving blocks vary, the width ranging from 2.75 to 3.94 ins., the length from 7.09 to 11.81 ins., and the height from 4.72 to 7.09 ins. The so-called Paris standard dimensions are 2.95x8.86x5.9 ins.

To prevent decay, the wood blocks are impregnated with a zinc-chloride solution or with creosote oil. The zinc-chloride solution contains 2 parts by weight of chloride of zinc, $ZnCl_2$, 3° Baumé, and 30 parts by weight of water. The creosoted oil should not contain less than 5 per cent of creosote and not more than 15 per cent of naphthalene and is distilled between 180° and 270° C. (356° and 518° F.). The specific weight of the oil is 1.07. The creosote-oil impregnation is considered preferable, because it also serves to forestall the subsequent swelling of the blocks by dampness after they have been set, and the resulting bulging of the pavement. To impregnate with creosote oil, the blocks are dried out for 3 to 4 hours in a tank or boiler at a temperature of 100° to 140° C. (212° to 284° F.), when they are transferred to another tank and allowed to remain for an hour in a vacuum of one-tenth to one-fifth of an atmosphere, after which creosote oil that has previously been heated to approximately 50° C. (122° F.) is pumped in and forced into the wood under a pressure of 6 to 8 atmospheres.

In constructing the wood pavement, the lower sides of the impregnated blocks are first dipped into hot tar or asphalt and then laid in a carefully prepared level layer of concrete 6 to 8 ins. thick. The blocks are set side by side close to each other, although a space of 0.079 to 0.197 in. is left at the ends between the rows. This space is filled with tar, and in some instances with asphalt. When asphalt is used, the intervening space is twice as large as when tar is the binder. To gauge the space between the ends, wood battens of the proper thickness are used. The rows of blocks may be set either at right angles with the axis of the street, or at an angle of 45°.

After the pavement has been laid, the surface is covered with a thin layer of asphalt or tar, over which a layer of coarse sand or fine gravel is spread. This, when pressed into the blocks, forms a durable coating which serves to prolong the life of the pavement. The sand or gravel is usually spread on the pavement once a month.

If the pavement is kept clean and if good materials have been used in construction, under ordinary conditions of traffic and of weather the surface wears at the rate of about 0.2 in. annually. Wood block pavements on an average last 10 to 15 years, but in dark, damp places, not exposed to sunshine, the blocks will decay in half the time. The approximate cost of block pavement in Berlin is 14 marks per square meter ($2.79 a square yard).

General

Report Recommending Methods of Collection and Disposal of Municipal Refuse of Chicago.

A report dated March 25, 1914, was made to the Chicago City Waste Commission on the collection and disposal of the city's refuse by Irwin S. Osborn and John T. Fetherston. The present article is based on that report and describes the scope of the investigation, the recommended project, gives notes relating to estimates for the various projects considered, states the general recommendations made and gives the estimated cost of the recommended project. If the project recommended is adopted and put into effect we shall describe it in more detail.

SCOPE OF THE IRRIGATION

In order to determine the most sanitary, economical and feasible plan or method for the collection and disposal of refuse in Chicago, a study was made covering the following items:

1. Topography of the city, with reference to its influence on collection.

2. Inspections of all parts of the city to determine the variable elements having a bearing on collection, also the conditions of house treatment of refuse.

3. Inspection with reference to possible locations for disposal plants or central loading stations, from which refuse could be transferred to points for disposal.

4. Inspection of the various wards to determine the present practice in making collections and work actually being done, with reference to improvements from a sanitary and economical standpoint.

5. Study of data submitted, and on file in the Bureau of Streets and with the Civil Service Commission.

6. Study of street cleaning activities now conducted by the Bureau of Streets, with reference to its influence on the collection of refuse.

7. Time studies of work in the field to verify assumptions made.

8. Comparison of available data for Chicago with that from other cities, with allowance for variation in local conditions, to determine the approximate quantities of each class of refuse that would be produced per annum or per capita per annum.

9. Study of past growth of city, with reference to future growth. Compilation of charts and tables relative to the same, and its relation to each ward. Estimates of population for each ward, as now established for years 1910, 1915, 1920, 1925 and 1930.

10. The results of the studies were applied to several projects. For each project there were assumed certain points of disposal or loading stations which might become available for the purpose desired. For each possible location, collection data were assumed, and a complete project developed.

The following statistics were determined for the years 1920 and 1930.

(a) The population to be served in each collection district, with reference to the shortest haul.

(b) The estimated quantities of each class of waste produced from each collection district.

(c) The estimated maximum quantity of each class of waste produced during the season of maximum production in each collection district, in order to determine the various plant capacities.

(d) The final capacities of disposal plants for each district, based on the estimated maximum production for the year 1930.

(e) The ton miles haul for each class of waste.

(f) The estimated cost of collection for each district, and for various methods of collection.

(g) The estimated capital costs of each project.

(h) The estimated operating cost for each disposal plant, for each project.

(i) The estimated annual revenues, for various methods of disposal.

(j) The estimated fixed charges for interest and depreciation.

(k) The net annual cost for each project.

(l) The estimated annual per capita cost

EXISTING CONDITIONS AND METHODS.

The collection and disposal of refuse in Chicago has been carried on under the direction of the Bureau of Streets, through the various ward organizations, each division conducting the work independently. Until last September garbage was reduced by contract. Since then it has been disposed of by dumping in clay holes as a temporary method of disposal. Before the contract expired with the reduction company garbage was collected by the city and delivered to the contractor's plant by the wagons direct, or by barges from loading stations on the Chicago River. The city owns the collection wagons and teams and drivers are hired by the city to make collections under the direction of the various ward superintendents. The population in 1910 was 2,062,889. The estimates for 1920 and 1930 are 2,905,531 and 3,503,521, respectively. In 1913 the amount of garbage collected amounted 98 lbs. per capita per year. The corresponding figure for ashes and rubbish was 555 lbs.

NOTES RELATING TO ESTIMATES FOR THE VARIOUS PROJECTS CONSIDERED.

Collection.—The cost of collection was computed by taking the sum of the estimated cost of loading and the cost of haul. The cost of loading was estimated for either combined collection or separate collection, according to the method of disposal. The cost of haul was determined from the ton miles of haul of each class of refuse collected separately or combined. The ton miles haul was computed by multiplying the estimated tonnage collected in each district by the average haul to the point of disposal or loading station.

To determine the average haul, the distance was measured from a point midway between the center of area and the center of population of each collection district following right angle street lines to the point of disposal or loading station. The cost per ton mile haul was computed by allowing a travel of three miles per hour per team; the cost of team and driver estimated at 75 cts. per hour, with allowance for time to discharge load. The estimates were verified by time studies and cost data available for similar work.

The cost of collection was computed, assuming a uniform production of each class of waste. This will not hold true in actual practice, due to various reasons, such as character of population or other factors that would effect the production in various districts. The uniform production was assumed for each project considered, so that the relative results would be constant.

Loading Stations. The loading stations were fixed at locations most suitable for the purpose, and as many as possible were taken at points recommended in the report of the Civil Service Commission, addressed to the Commissioner of Public Works, under date of Oct. 31, 1913, and published in abstract in Engineering & Contracting of Nov. 26 and Dec. 3, 1913.

The cost of land was taken from the Bureau of Efficiency Report, but the cost of buildings and appurtenances was increased to provide modern equipment and more permanent construction. The studies have not contemplated the utilization of loading stations for other purposes than refuse handling, but in most cases they will be found of sufficient capacity to care for the handling of other waste, such as street sweepings and catch basin cleanings. If the loading stations are utilized for handling trade wastes or building refuse, a charge should be made for this purpose.

Trolley Transportation. The estimates for street railway cars were based on the number required to move the material from the various loading stations, to the points of disposal. The cost of transportation by street cars was based on the present cost for such service in Chicago.

Barge Transportation. Barge transportation was estimated by taking the cost of maintaining and operating the necessary equipment, based upon existing data in Chicago.

Fixed Charges.—Interest was assumed at 5 per cent, and depreciation was based on a sinking fund, which would equal the cost of the plant during the estimated life of the various elements making up the installation, the interest being assumed at 5 per cent.

Operation of Incinerator.—The cost of operating incinerators was based on the cost of labor, repairs and attendance on electrical equipment per ton of material handled for the year 1920.

Operation of Reduction Plant. The cost of operation of the reduction plant was based on the cost of overhead charges, labor, fuel, supplies, repairs and renewals required to dispose of quantities produced in 1920.

Capital Costs of Incinerators.—The capital investment for incinerator plants was based on current unit prices for entering into the construction of an incinerator of average capacity for the number proposed.

Capital Cost of Reduction Plant.—The estimated capital cost of the reduction plant was based on a detailed estimate for building and equipment.

Truck Costs.—The estimated cost for auto truck haul was based on data for similar work, and the same class of material.

Capacity of Disposal Plants.—The capacity of the various disposal plants were based upon the estimated maximum production of the different classes of refuse, for the year 1930.

Building for Disposal Plants.—The estimated cost for the construction of buildings contemplated the erection of fireproof buildings of plain design, yet so proportioned and constructed as to be of pleasing appearance.

Machinery and Equipment.—Estimates for machinery and equipment for the various disposal plants include the most modern types for the work intended.

Labor Cost.—Labor costs were computed on the rate of wages paid for similar work in Chicago.

Revenues.—The revenue to be obtained from the operation of disposal plants consist of power and clinker from the incinerator plants and the by-products (grease and tankage) produced by the reduction method. Quantities are estimated for the year 1920.

Power.—The value or credit allowed for power developed by the incineration of refuse was based on the amount and availability of power for the uses intended.

Clinker.—The value of clinker was based upon current prices for similar or equal materials used for pavement foundations, sidewalks and similar concrete work.

Grease.—The grease recovered from garbage was estimated to be 3.25 per cent of the total weight of the garbage as delivered.

Tankage.—The solids or tankage recoverable from garbage were estimated to be 11 per cent of the total weight of the garbage as delivered.

Value of By-Products.—The value of by-products fluctuate with the supply and demand for products of that nature. Tankage is used in the manufacture of commercial fertilizer and sold on the basis of its analysis of fertilizing elements.

The average price for tankage will vary from $6 to $10 per ton and in estimating revenues from disposal by reduction, the price was taken at $7 per ton.

The value of garbage grease likewise fluctuates with the supply and demand. The average price that has been received for garbage grease during the past seven years is approximately 4.13 cts. per pound.

In estimating the value of grease for Chicago, the price was taken at 4 cts. per pound.

RECOMMENDATIONS

As a result of the studies and investigations the following recommendations were made:

1. That the city should own and operate a complete refuse collection and transportation equipment, also refuse disposal works.

2. That regular and systematic collection of separated classes of wastes (ashes, garbage and rubbish) be made at daily or tri-weekly intervals, depending on the character of the districts served and the seasons of the year.

3. That the laws regarding house treatment in respect to the separation of the different classes of waste be strictly enforced.

4. That separation of all classes of refuse be made by the householders, except in districts where the combined refuse is disposed of by incineration.

5. That separated garbage be treated by the reduction process at a central plant, located for service by barges to be used in transporting the material from waterfront loading stations, except such garbage as may be economically hauled by wagon or truck direct to the plant.

6. That separated ashes be disposed of by filling low lands or depressions in need of grading, and as far as practicable, that such lands be purchased by the city, so as to secure the benefit from the enhanced value derived from the improvement.

7. That small incinerators be constructed at each loading station for burning the separated combustible rubbish. Other classes of rubbish, including metals, glassware, etc., should be reclaimed at the loading stations and sold. The unsaleable and incombustible rubbish should be disposed of with the ashes.

8. That the present garbage loading stations be remodeled and that at least three additional garbage loading stations be provided on the river or canal.

9. That a loading station for ashes be provided in each district where street car transportation is more economical than direct team haul to dumps.

10. That garbage receiving stations for motor trucks be provided to reduce the team haul.

11. That a modern high temperature refuse incinerator be installed at Stony Island Ave. and 95th St. to dispose of the combined refuse from that district.

12. That the mechanical analyses and tests of refuse started early this year be continued for a period of at least one year or longer, to determine the seasonal variation of the several classes of waste.

13. That a competent technical staff be employed to develop, install and operate for at least one year the project herein recommended, and to make such further studies and tests necessary to determine in detail the most suitable types of receptacles and equipment for a model collection service.

14. That the maintenance division in charge of the collection and disposal systems installed, be provided by the technical staff with carefully determined standards of performance and unit costs, in order that proper control may be exercised over the work.

15. That $3,513,000 be provided for the purchase of collection and transportation equipment, and the construction of reduction works and incinerator plants. Of this sum, $85,000 should be made available for the first year's expenses of the technical staff.

BRIEF DESCRIPTION OF RECOMMENDED PROJECT.

This project consists in the disposal of garbage in a central reduction plant, and the disposal of rubbish in small incinerators located at each loading station, except in the district comprising the 8th and 9th wards, which is provided with an incinerator for total incineration of all garbage and rubbish. The city is divided into 11 collection districts, with a loading station in each district, for transfer of the material by trolley, barge, auto truck or wagon. It contemplates the separate collection of ashes, rubbish and garbage, except in districts where total incineration is contemplated; all ashes are to be disposed of by fill.

ESTIMATED COSTS

The following tabulation gives the estimated capital cost, annual expenditure, revenue and net annual cost for the recommended project:

Capital cost		
Collection equipment	$[illegible]	
Stables	[illegible]	
Loading stations	[illegible]	
Street cars	[illegible]	
Water transportation equipment	[illegible]	
Motor trucks	[illegible]	
Incinerator	[illegible]	
Reduction works	[illegible]	
Total capital cost		$[illegible]
Annual cost (operation, maintenance and fixed charges)		
Collection	$1,[illegible]	
Loading stations	[illegible]	
Street car transportation	[illegible]	
Water transportation	[illegible]	
Motor truck transportation	[illegible]	
Incinerators	[illegible]	
Reduction works	[illegible]192	
Total annual cost		$[illegible]
Revenue		
Reduction works (grease and tankage)		[illegible]654
Net annual cost		$1,[illegible],107

PROSPECTIVE BETTERMENTS UNDER NEW PLAN.

Under the recommended project the service to be rendered the people of Chicago will be increased approximately 50 per cent as regards frequency, regularity and amount of collections. The cost of collection, transportation and disposal of refuse under existing methods for the year 1912 was approximately $1.87 per ton.

In 1920, by the recommended methods, the cost per ton is estimated at $1.48. This represents a saving of 39 cts. per ton, or over $500,000 for the amount (1,307,490 tons) of materials collected and disposed of in 1920, when compared on the same basis with the year 1912.

A Specific Suggestion Looking to the Improvement of the Material Status of the Engineer.

To the Editors: The issue of Engineering and Contracting for Feb. 11, 1914, contained an extremely interesting article entitled "The Functions of the Engineering Society," by Mr. L. K. Sherman, president of the Illinois Society of Engineers and Surveyors.

In one part of the address the remark was made that "The technical press of the country during the past year contains more communications than in any previous period on such subjects as 'The Staus of the Engineer,' 'Compensation of Engineers,'" etc. The evident interest manifested in such discussions prompts me to indorse heartily the greater part of Mr. Sherman's address and leads me to believe that some amplification of points raised by him would be of interest to your readers.

It is certainly a most hopeful sign of the awakening of the engineer from the dream of the false hope into which he has been soothed when at school and college for the purpose of closing his eyes and ears to the hard facts to be faced in the life ahead of him, to see such statements made in a presidential address before an engineering society as: "It is, however, not our place merely to accept what the conditions of the times offer; it is our duty to take advantage of and to push those conditions which result in our rightful material welfare. This can be accomplished only through organization."

There are in this country four great national engineering societies representing the recognized branches of engineering, and 200 or more very much smaller societies, the objects of which are similar but with a sphere of activity limited to a state, city or locality. Mr. Sherman suggests that the time has come for these societies, the objects of which are, in general to advance science and engineering, to enlarge their field of endeavor and take up the problem of advancing the interests of the engineer.

The first point I would like to take up is the suggestion "to enlarge the field of endeavor" of the present societies.

It is possible that there would be no great difficulty in carrying out this suggestion in many of the small societies, most of which have a membership of considerably less than 1,000. It is quite another matter with the large societies which in addition to converting their membership of 5,000 to 10,000, have also quite a load of tradition to shake off before such action could be taken. That this is so may be readily seen from the admitted fact that there is a very active demand for something more than these bodies offer, from the average member or prospective member, yet at the last annual meeting of one of them, the American Institute of Electrical Engineers, a paper was read proposing the formation of a United Engineering Society, composed of the membership or part of the membership of the big societies. Three considerations of particular interest may be deduced from this. First, it is an admission of the failure of the present big societies to answer all the requirements or supply all the needs of the persons composing the engineering profession. (Hasty readers are requested to note that this casts no reflection on these bodies, for they have nobly lived up to and filled their professed aim, a fact to which the many wonders of engineering of the last quarter of a century amply bear witness.) Second, it is an indication of the belief in the minds of those advocating the move that it would be easier to create such a new organization than to alter the existing ones. Third, that it is a sub-conscious acknowledgement of the fact that while the best interests of engineering may be advanced by subdivision and specialization, the best interests of the individual, the engineer, must be advanced by consolidation, by unification. There is considerable ground then to doubt the practicability of enlarging the field of endeavor of the great national engineering societies.

To return to the consideration of the more flexible small societies. While the great number of them indicates that the tendency toward organization is very general, yet the small numerical strength of most of them indicates that they are too weak to accomplish any great material advantage for their members. It must be remembered that so long as a small society has for its object the advancement of science and engineering it will never meet with opposition, but on the contrary it will receive all sorts of encomium for its endeavors. Let that same society start to advance the material interests of its members and it will not get very far before it finds opposition which it would be extremely costly, probably prohibitively so, for any but a strong organization to make headway against. Mr. Sherman states that his society is *honored* by having *one* member on the State Public Utilities Commission. The governor of Illinois, upon *petition* of the State Society of Engineers, appoints *one* engineer to the Public Utilities Commission and as a consequence the society is *honored*, and no doubt all Illinois is happy. Let the society petition the governor to appoint at least a majority of engineers to the Public Utilities Commission, as should in common sense be done, and to other posts which should be held for the public interests by competent engineers, then those who now hold those positions and would stand to lose them will be ranged in active political hostility to the society. It must be plain that no small society could stand the financial strain such activity would call for in addition to the other expenses such as publications it has to face. The proposition facing local or state societies is then one of acquiring sufficient strength to carry out a policy of material benefit, and the question is, Can they get that strength?

From the records obtainable it does not appear so, and if we are genuine in our desire to organize to our best possible advantage we must criticise freely and to the point and face facts as they are. The best answer that can be given to such a question must be based on results and by way of illustration we will take one civil engineering society which has been in existence over a quarter of a century in one of the most progressive states in the Union. This body has succeded in that time in gathering less than nine per cent of the civil engineers of the state, according to the figures issued by the latest census. This may be due in great measure to the fact that the organization in the declaration of its objects adheres practically to the stereotyped ones of advancing science, etc. There can be little doubt, however, that a very general reason is that pointed out by Mr. Sherman when he says: "One of the unavoidable handicaps of the civil engineer arises from the very nature of his work. His work ends when the particular structure which he may be engaged upon is completed. He then must seek other and often remote fields where new construction is to be started. Thus the engineer is a migratory animal." This being the case, it is unnatural to expect a man to support with any deep enthusiasm a local society to force beneficial legislation in one state and then be compelled to pass on to another remote state, lose all his associations, go outside the sphere of influence of the society he has been supporting, lose his membership and money only to start the same thing over again in each state he is driven to in search of employment.

A comparison of the memberships of local societies makes the force of this argument irrefutable, more especially when we find that the large national societies have bigger memberships in the respective states than the state societies.

What then is the situation?

First, the members of both the large and small societies are asking for more consideration for the individual. Second, the members of the large societies themselves appear doubtful of either the propriety or possibility of broadening their scope. Third, the smaller societies appear to be unable to build up a membership such as would warrant the adoption of aggressive tactics.

What then is to be done?

We have seen that the unity of the economic and social interests of engineers in all branches of engineering is understood. We have seen that the vitality of local organizations is a minimum.

The logical deduction, then, is indisputably: To form one large national organization, of which the avowed object shall be the raising of the professional and social status of those who earn their living by serving the community in the various fields of engineering.

It would require considerable space to enumerate all the advantages of such a course, so only two or three of the more prominent will be stated.

First, the local societies could become branches of the main body; they could retain their own individuality and be self-governing. Second, instead of spending their slender incomes on the publication of quarterly or, in some cases, annual "transactions" they would receive monthly or even weekly the magazine of the main body, and in this way receive more information and be able to spend more money locally for purposes pertaining to the material advancement of members. Third, thousands who will not now join local bodies nor national bodies of almost purely scientific aims, would immediately join an organization established to advance their interests, when they know that a matter of changing from one state to another would merely be a case of obtaining a transfer to another branch of the same body.

We are not without practical demonstration of the correctness of this policy, because several attempts have been made to organize draftsmen but only one has proved entirely successful; that is the American Society of Engineer Draftsmen. The reason for this success has clearly been that the organization is national in scope and broad enough to take members in every branch of engineering and architecture. Even in this case it has been amply proven that the success would have been more rapid had not the error been made of emphasizing one grade of engineer—draftsmen—instead of avoiding the inferred restriction by the adoption of a more comprehensive name. So increasingly evident has this become that a suggestion to alter the name to one better fitting the composition of the membership, among whom are engineers, surveyors, superintendents and other grades, is now under favorable consideration.

The growth of the American Society of Engineer Draftsmen has been phenomenal in engineering circles, for it has in less than four years gathered a membership nearly one-fifth of that of the national societies in their thirty-odd years of existence. It is a foregone conclusion that if the change of name is adopted the growth will be more rapid than ever.

There are several other points in Mr. Sherman's address which would be more than interesting to discuss in view of the attention being given to the subject at the present time, but it is hoped that these remarks will help to make clear that the first function of an engineering society, the aim of which is to raise the material status of its members, should be to foster the interests of all, not merely those in a certain locality nor those belonging to one or two grades of the profession. The draftsman of today is the engineer of tomorrow. The man who is working in New York today may be compelled to seek work in Seattle tomorrow. Let us face facts as they are and get together on that basis.

Very truly yours.
WALTER M. SMYTH.
General Secretary, American Society of Engineer Draftsmen.
74 Cortlandt St., New York City, Feb. 28, 1914.

A New Gasoline-Operated Pulsator Air Drill.

(Staff Article.)

The air drill operated by a pulsator actuated by an electric motor as illustrated by the sectional view, Fig. 1, is a standard apparatus. A modification of this apparatus consisting of the substitution of a gasoline engine for the electric motor has recently been placed on the market and the new gasoline pulsator unit is illustrated by Fig. 2. The following description is from an article prepared by the manufacturers, the Ingersoll-Rand Co., New York City:

The gasoline engine is of the jump spark type, the ignition spark being obtained from dry cells. The circulating water is obtained from any convenient receptacle placed near the equipment. The splash system of lubrication is employed for the piston and crank pin bearing and grease cups lubricate the main bearings. A gasoline supply tank of 1¼ gals. capacity surmounts the engine. The fuel consumption of the engine, running under load, is about 2 quarts of gasoline per hour, so that the average daily fuel consumption would be approximately 3 or 4 gals.

The drill proper of the "gasoline-air" unit is driven by pulsations of compressed air created by a pulsator actuated by the gasoline motor. Gearing transmits the power from motor to pulsator. The air is never exhausted, but is simply used over and over again, playing back and forth in a closed circuit. The pulsator is a simple machine, employing no water jackets. The drill is the simplest type possible—a cylinder containing a moving piston and rotation device, with no valves, chest, buffers, springs or side rods. The cylinder is larger, but the piston portion is shorter, making the weight of the drill unit about the same as, or even less than, that of the corresponding air drill. Two short lengths of hose connect pulsator and drill, each hose acting alternately as supply and exhaust.

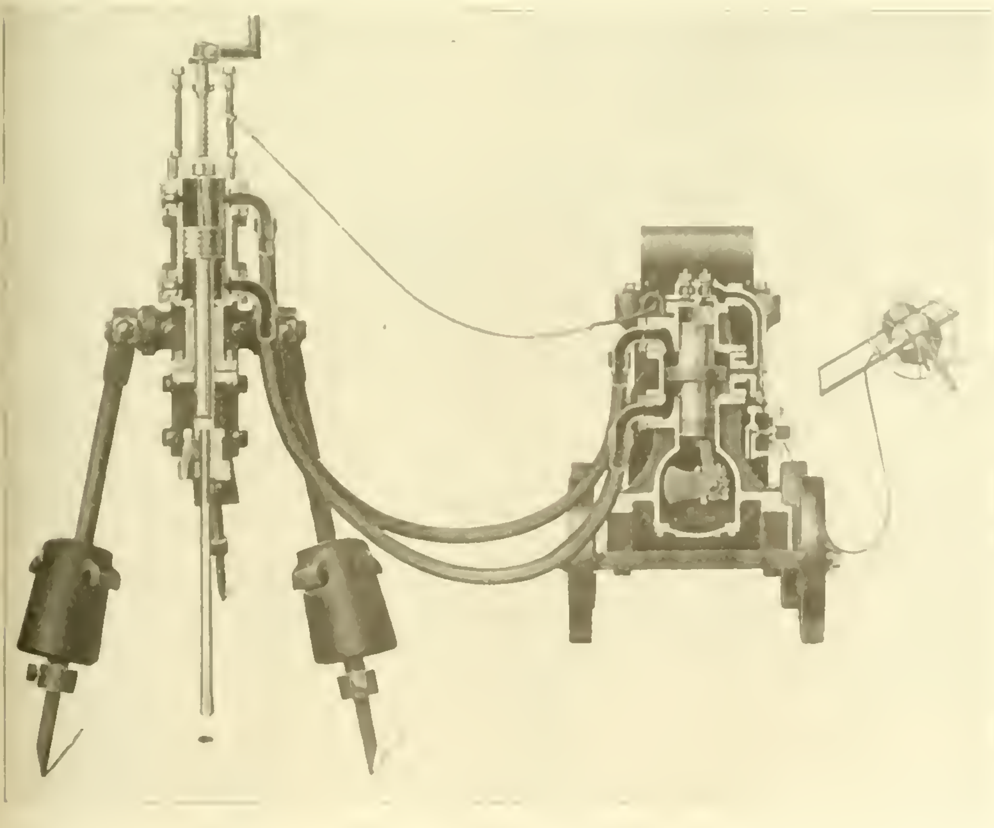


Fig. 1. Electric Operated Pulsator Air Drill.

Some leakage of air from the system is inevitable. This is provided for by a compensating valve on the pulsator which is adjusted to automatically maintain the requisite pressure in the circuit.

The ordinary air or steam-driven rock drill takes a full cylinder of air or steam at full pressure each stroke, and discharges it to atmosphere at practically full pressure. No advantage, therefore, is taken of the expansive properties of the air or steam, and as a result an amount of power is wasted without doing useful work. The "Gasoline-Air" drill operates on an entirely different principle. The closed system is filled with air under a low pressure, which is simply an agent for transmitting the effort of the pulsator piston to the drill piston. The air in the system has been aptly referred to as a pneumatic "spring," unwearing and unbreakable, exerting its pressure on opposite sides of the drill piston and the pressure in the air is analogous to the tension of a spring. That the saving in power is great is proven by the fact that, under ordinary conditions, the drill proper of the gasoline-air unit uses about one-fourth the horsepower required for the usual air or steam drill of the same work capacity.

The gasoline-air drill has a stroke equal to, or even greater than, that of the air-driven rock drill of corresponding capacity. The length of stroke is varied simply by cranking forward in the shell, and both stroke and force of blow may be adjusted by the same means for fast drilling under any circumstances. If a hole should "mud up" or form a "mud collar" in bad rock, the machine can be backed out without injury while running, thus clearing itself quickly. The cushioning is such that the piston, in running, does not normally strike either front or back head. This makes very easy the problem of handling it in all kinds of drilling.

When the ordinary rock drill, whether steam or air driven, sticks or fitchers it simply pulls back with a steady pressure and the steel must be sledged until it loosens. The gasoline-air drill, on the contrary, when it does momentarily stick, receives on its piston upwards of 100 alternating pulls and pushes per minute, and this repeated pulsation has a tendency to promptly loosen and dislodge the stuck bit.

The system of lubrication of the pulsator is automatic, the "splash" method being employed. While most of the oil drains back to the crank chamber, a portion is atomized and carried through with the air into the drill.

The drill cylinder diameter is 4¾ ins. and the stroke is 7 ins. The drill will accommodate octagon steels from 1 to 1⅛ ins. in diameter, drilling holes from 1¼ to 2 ins. in diameter. The drill feed is 24 ins. The approximate strokes per minute are 110. The machine is designed to drill holes up to about 12 ft. in depth.

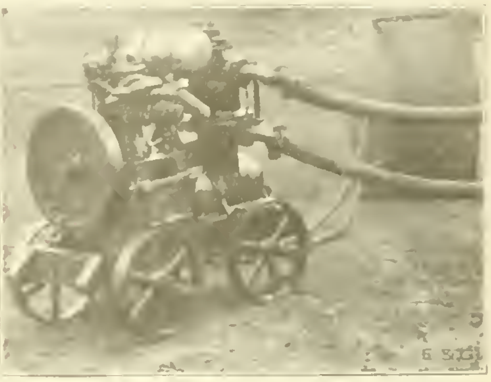


Fig. 2. Gasoline Engine Operated Pulsator for Air Drill.

On the Patenting of Engineering Ideas.

TO THE EDITORS: The writer was greatly impressed with the excellent article of Mr. John W. Alvord, entitled "The Consulting Engineer in Water Works Practice," which appeared in your issue of Feb. 18, 1914.

There is one sentence, however, which in itself is all right but suggests a rather unfortunate attitude possessed by many engineers. He states:

> Very few consulting engineers have patented their ideas extensively, and remained consulting engineers.

This is an implied hint that the United States patent laws are subversive of public welfare. The writer does not share this view.

In every country in the world it has been recognized that mankind needs material stimulus to put forth their best efforts.

Human progress is measured by invention and discovery. If there were no patent subsidies there would be fewer inventors. Some might hold that all the important and great inventions would be made anyway without this stimulus.

From a monetary standpoint the successful consulting engineer is probably the man who follows best the beaten path established by his contemporaries or predecessors. In steam engineering this man was thoroughly familiar with all the knowledge and best practice up to date, until the engineer, George Corliss, designed or invented a little trick in valve gear that revolutionized the practice and made it possible to do the same work with half the amount of coal. No doubt, the activities of Mr. Corliss were frowned upon by many of the prominent engineering contemporaries of his time. Nevertheless, his achievement made him as an engineer worth more than a hundred of the class that follow only the beaten path, and properly so. He no doubt expended more intrinsic mental effort and vitality on that invention than other engineers might spend in their whole life time. Unfortunately, there are thousands of inventions that are very meritorious as mental achievements, but still have no commercial value. This simply means that the inventors have utilized their efforts in unworthy fields.

It is probable that some of the odium to which patenting has been subjected is due to the defects in the patent laws which are difficult to overcome.

The length of time the patentee is protected

(17 years) cannot be considered unduly long, not more than a third of a generation. The invention then belongs to the public, but unfortunately, the laws are such as to make it possible and not unusual, that during this time the art may be absolutely suspended without benefit either to the inventor or the public. One man may discover a basic idea of great public value and have it patented but there may be some undiscovered feature whereby it cannot be put into practical use. Another man overcomes this difficulty and the two inventions combined are immensely valuable, but the owner of the first cannot use the invention of the second except with his consent and vice versa. This might be remedied by amending the law so that no inventor could exact for his invention a greater royalty than a certain percentage on the cost of labor and material entering into such invention, or by a provision whereby a certain specified court will upon application establish the royalty for each case under consideration.

If the consulting engineer is frank with his client and informs him that certain devices which he has recommended are by him patented, no injustice can be done.

The art of invention is the greatest of all and it is one in which "many are called but few are chosen." It is easy enough to invent mediocre things, as is evident in any field, like that of music. Beethoven and Bach invented melodies and harmonies that thousands have tried to equal in quality without success, but the great ability in achievement is seen when it is known that Beethoven's finale to his Kreutzer's Sonata was worked on for 20 years before he was satisfied with it. The time is coming, if it has not already arrived, when engineers will be classified according to the quality or number of meritorious discoveries and inventions which they have accomplished.

There are many inventions and discoveries that do not lend themselves to patent. The engineer that suggested the "mass diagram" for computing the yield of streams by storage was a true inventor. The man who discovered the method of solving cubic equations was a true inventor, and so was the man who discovered the law of gravity and the law of evolution, and the law of gases, but these, like many other well-known engineering achievements, can not be covered by the patent laws, and the discoverer must necessarily, even without philanthropic motives, donate them to the public.

Men like Edison and Westinghouse, engineers of the highest type, have not hesitated to take advantage of the patent laws; and in fact we think that this is the case with nearly if not all engineers who are capable of producing useful patentable inventions.

Therefore, if consulting engineers should formulate a code in line with Mr. Alvord's suggestion, that "No engineer shall be eligible as a consulting engineer who is the proprietor of a patent right," the public might thereby be deprived of the services of men of the highest special training.

Very truly yours,
"A PATENTEE."

Philadelphia, Pa., March 7, 1914.

(The foregoing communication has been brought to the attention of Mr. Alvord and his reply follows.—Editors.)

To the Editors: I have been handed a letter to your journal, signed "A Patentee," commenting on my remark as to consulting engineers and patents, and am glad to amplify my thought on this point a little more.

In the first place, it will be noted that the statement is qualified. It said "*very few* engineers have patented their ideas extensively and remained consulting engineers." There are undoubtedly important exceptions to this general remark, and they are important because the character and standing of the patentee has been so great and the value of his ideas so marked, that by these means the ordinary hamper to successful consulting practice has been overcome. As a rule, however, with the average engineer, the knowledge on the part of his clients that he has patented his ideas, undoubtedly causes excessive caution, and often leads to the discounting of his opinions by the public to a greater extent than is sometimes warranted where his patents are concerned. An engineer usually patents ideas which lie along his special and most extensive researches, and, therefore, are in the line of his greatest usefulness. If these patents are not valuable they are quickly forgotten, and he is not embarrassed by their existence, but if they are valuable he is tempted to be led into the manufacturing business, and to become commercialized along those lines in which he is probably the best informed, and in which his ideas as a consulting engineer are the most valuable.

It is not necessary for engineers to provide any code of ethics on this subject, because it settles itself. Consulting engineers, as a rule, are so much more injured than helped by patenting their ideas, and clients are so apt to be sensitive on this point, that self-interest alone will caution the consulting engineer to be careful how he patents ordinary ideas extensively. Only patents likely to be of the most wide use and of the greatest value can be safely patented by the consulting engineer, and then with doubtful help to his consulting work.

The writer has not relied upon theory alone in this opinion, but has for years watched the effect of patents upon the career of those engineers who desired to achieve a reputation as independent advisors, and in almost every case which he can recall, either such engineers have become promoters and manufacturers, in large part, or else they have ultimately quietly set aside their patents and devoted their entire attention to the consulting field. The writer is willing to confess that he is among this latter number, and that after having taken out five patents he found he had to face this problem, and having thought it out along the lines here indicated, the result has been that one patent was sold, two others, covering ideas now in successful use, have been filed away with the definite purpose of entirely forgetting their existence, and the other two have relegated themselves to innocuous desuetude because they were of no very great value.

One can only say that to those eminently successful men who have both patented their ideas and remained consulting engineers, great deference is due, because it undoubtedly implies a high character and an ability greater than the ordinary.

Very truly yours,
JOHN W. ALVORD.

Chicago, Ill., March 31, 1914.

A Comparison of Costs of Wood and Coal Used as a Fuel for Construction Plant.

Contributed by J. R. Sherman, Assistant Engineer, U. S. Reclamation Service, Meadow Creek, Wash.

Kachess Dam, which was built in 1911-12 by the United States Reclamation Service, is located at the lower end of Lake Kachess, near Easton, Wash. During the construction of this dam both coal and wood was used extensively as fuel on practically all of the dirt handling machinery. The work being located in a heavily timbered forest reserve wood appeared to be without question the cheapest fuel to use, since no charge was made by the Forest Service for the timber so used. Twelve miles from Easton, however, the Roslyn coal mines are located and from which a large part of the bituminous coal burned in the northwest is mined.

TABLE I.

Machine—	—Fuel used—	Total cost.	Cu. yds. excavated or hauled.	Cost per cu. yd.
Steam shovel	Wood, 16-in., 36-in., 42-in.	$1,275.45	58,799	$0.022
Steam shovel	Coal	1,023.36	82,918	0.012
Lidgerwood drag-line excavator	Wood, 36-in.	902.40	38,709	0.023
Lidgerwood drag-line excavator	Coal	321.72	19,596	0.016
Dinkey engine shovel run	Wood, 16-in.	491.77	57,719	0.0086
Dinkey engine shovel run	Coal	423.12	72,221	0.0059
Dinkey engine excavator run	Wood, 16-in.	182.20	11,610	0.0157
Dinkey engine excavator run	Coal	393.60	44,544	0.0088
Steam roller	Wood, 16-in., 36-in	275.11	31,750	0.0087
Steam roller	Coal	300.12	77,673	0.0039

The wood used was cut by contractors on individual contracts for $1 for 16 in., $1.25 for 36 in. and $1.40 for 42 in. wood per rick, a rick being 4 ft. high by 8 ft. long; the only requirements being that the wood should be cut as near to the work as possible and piled so as to admit of easy access for the wood wagons.

The wood was distributed to the various machines by Government forces, the average amount hauled with a team, wagon, driver and one extra man being 2½ ricks of 16 in. wood, 2 ricks of 36 in. wood and 1½ ricks of 42 in. wood. The length of haul varied from a few hundred feet to approximately one-half mile, the roads for the greater part of the season being in good condition.

Mine run coal was purchased from the Roslyn Fuel Co., Roslyn, Wash., the contract price being $2.75 per ton f. o. b. Roslyn. At Easton the coal was sacked by hand and hauled by team and wagon from Easton to the works, a distance of three miles. The average load for a four-horse team was three tons.

The itemized cost of the coal delivered at the machines is given below:

Item—	Per ton.
Purchase price	$2.75
Ry. freight, Roslyn to Easton	0.37
Freighting from Easton to camp	0.90
Sacking and storehouse expense	0.38
Distributing at camp	0.52
Total	$4.92

The unit cost of distributing at camp as shown above was obtained by dividing the total cost of distribution by the total tons used. There was a large amount of coal used that did not have to be distributed, the coal being unloaded near the tracks so that the dinkey engines used to haul the dirt trains could coal up from the storage bins.

A comparison of the costs of fuel used on some of the machines during the season of 1912 is given in Table I.

It should be stated that while the machines were using wood it was necessary to keep an extra man employed for splitting. On the dinkeys and steam roller this wood splitter was not used all the time, as one man was able to split wood enough in 8 hours to run the machines 16 hours. On the steam shovel and drag line excavator a wood splitter was employed for each shift, the wages being $2.20 for 8 hours' work.

The steam shovel used was a 45-ton Bucyrus with a 1½ cu. yd. dipper and was used to excavate the finer material for the embankment, working in a pit approximately 1,300 ft. long and with an average depth of cut of 20 ft.

The drag line excavator was manufactured by the Lidgerwood-Crawford Co., and was used to excavate gravel for the embankment. It was a 65-ton machine with a 70 ft. boom and a 1½ cu. yd. Page bucket.

The dinkey engines were made by the Vulcan Iron Works and were operated on a 24-in. gage track and weighed 9 tons each. The average length of haul for the dinkeys operating on the steam shovel pit was 1,000 ft. and the average train was 15 1½ cu. yd. cars. The length of the haul for the dinkeys on the gravel run was about 3,000 ft. and the average train consisted of 12 1½ cu. yd. cars.

Approximately 12,000 cu. yds. of the steam shovel excavation was hauled by teams, which accounts for the difference in yardage excavated by the steam shovel and that hauled by the dinkeys on the steam shovel run.

Engineering and Contracting

Devoted to the Economics of Civil Engineering Design
and to Methods and Cost of Construction

Volume XLI. CHICAGO, ILL., APRIL 15, 1914. Number 15.

The Extent to Which Engineering Graduates Follow the Practice of Engineering.

Practicing engineers and others interested in engineering education who give the problem serious study are sometimes called upon to defend the work of our engineering schools. In view of the paucity of published data concerning the activities of engineering graduates it is only natural that misunderstandings should exist. We believe that a more general knowledge of the work of engineering graduates in practice, together with a certain amount of legitimate publicity, will do more at this time to advance the standing of engineers than any amount of discussion concerning compensation. The accompanying data concerning the present activities of the 2,165 engineering graduates of one of our leading engineering schools located in the Middle West are instructive and deserve attention. As architecture is taught in the college of engineering, in this particular school, the tabulation includes architectural graduates. The data immediately following refer to graduates now engaged in engineering and architectural work:

Classification—	Number.	Pct. of total.
Employees of engineering firms	1,370	63.2
Architects	318	14.6
Executive officers of engineering companies	130	6.0
Teachers of engineering subjects	95	4.6
Consulting engineers	20	0.92
Total	1,933	89.32

From the above table it will be seen that 1,933 of the 2,165 graduates, or 89.32 per cent are now engaged in engineering and architectural work. Of the total number of graduates 59 have died. To illustrate the kind of work now being done by the remaining 173 graduates and to demonstrate the fact that their work is important, the following classification is given:

Classification—	Number.	Pct. of total.
Farmers and ranchmen	52	2.39
Merchants	31	1.42
Executive officers of mercantile companies	19	0.88
Editors	19	0.88
Real estate and insurance	15	0.68
Salesmen	10	0.46
Army officers	8	0.36
Bankers	6	0.28
Physicians	5	0.24
Lawyers	5	0.24
Unclassified	3	0.13
Total	173	7.96

In the above quoted statement these men are classified as being engaged in non-engineering work, but a knowledge of conditions brings out the fact that some of the men classified as editors and salesmen are also engaged in engineering work. The number of men so engaged is small, however, and does not essentially affect the percentages given.

These data show that only 7.96 per cent of the total number of graduates are not following the kind of work for which their training presumably best fits them. We believe that this percentage is relatively low when compared with those for other lines of work requiring specially trained men. There is, of course, danger in drawing conclusions based upon data furnished by a single school, but the one here considered is fairly representative of the better class of engineering schools. Based upon past performances these schools have been quite successful in meeting the needs of the engineering profession.

As the possibilities of engineering here considered in its broadest sense are becoming better realized, we believe that our engineering schools will find it increasingly difficult to meet the situation successfully. They will need be even more energetic and alive to the development in the engineering field than in the past, and must find more certain means of attracting and holding the best in engineering if they expect to make as good a showing in the future.

The Position of the Engineer on the Question of the Method of Flood Control.

President Wilson has authorized the statement that legislation providing for Federal financing and direction of flood control will be urged upon Congress at its next session. Commenting on this announcement, the Washington correspondents of the daily papers mention that a serious obstacle to flood control legislation is the disagreement among engineers upon questions of methods. The feeling is strong that experts in stream hydraulics have divided into contending factions committed to radically different methods of solving the problem of flood regulation. This is unfortunate. It is unfortunate because it tends to confirm in its indifference to flood control a public whose apathy is the excuse of congress for inaction. It is doubly unfortunate because it is a belief based on impressions only partially supported by facts.

If we seek for sources of the belief that engineers disagree on methods of flood control we find that they are, mainly, two: One is the positive disagreement between engineers regarding the influence of forests upon run-off. The other is the apparent disagreement between engineers regarding the utility as a means of flood control of flood water detention reservoirs. The active and often acrimonious discussion by engineers of these two questions and the espousing vigorously by engineers of one or the other side of the discussion have been construed to have a broader significance than they really possess. Because engineers have disagreed whether forests do or do not materially retard the run-off into the streams of rain and snow, it has been concluded that they thus have opposite views of what, under conditions as they are, it is necessary to do to control floods. Because, also, engineers have, for specific flood works, taken sides for and against flood detention reservoirs, it has been concluded that they are divided into one party advocating reservoir construction in all cases, and another party contending against reservoir construction in any case

It is not difficult to see how the conclusions instanced have been come to, but the more important thing to understand is that they do not conform to the facts. And they certainly do not. There are, party to every dispute, extremists who demand all or nothing. Therefore we find here and there one who claims all merit for reservoir control of floods. And we have here and there one who grants no merit to reservoir control. These men do not truly represent engineers, and their extreme and divergent opinions are not the opinions generally of experts in stream hydraulics. Proof of this assertion can be found in the reports and writings on flood control of both the army engineers and of prominent hydraulic engineers in private practice. A dozen of these reports and articles have been reported within the year in these columns.

So also, when we consider the controversies between engineers on river control and improvement about the influence of forests on floods we find that it does not indicate a division of opinion on proper works for flood regulation. Patently in many places opinion on forest influence cannot have a part in the flood-control problem. The forests are gone in many states in which flood regulation is most demanded. Forests and their influence on floods, be it much or nothing, are not a part of the problem of planning flood control works in these states. No engineer, though he believes that, had the forests remained, floods would have been smaller, will hold his hand in doubt from designing flood control works for a stream whose drainage basin has been denuded of timber. Flood control works are designed to meet the conditions that are, and all engineers are in accord on this doctrine, however vigorously one may dispute with another what the conditions would be if lumbering had not cleared the land of trees.

Excluding here and there the extremist, engineers who have studied most flood control problems are convinced that they are not alike in any two instances and that a single method of solution for all is neither economic nor technically defensible. In any specific case of stream control there is no greater divergence of engineering opinion about methods than there is about methods of water purification or sewage disposal or shop arrangement. Likewise in many cases either of two or more possible methods of flood control will be equally effective and often not greatly different in cost.

There is nothing in this assertion of the position of engineers on flood control methods that is unknown to the expert in stream control, and nothing that will not appear reasonable and consistent to the engineer whose knowledge in this specialty is only incidental. The argument is presented, therefore, not to correct an error among engineers, but to point out the direction in which the engineer needs to advise the public. There are various engineering means for controlling floods. There are levee systems, channel enlargement and rectification, and detention reservoirs. No one means is the best under all conditions. Indeed the combination of two or more may be the wisest plan for a particular case. Legislation authorizing federal financing and direction of flood control cannot, therefore, wisely be legislation specifying a particular method of flood control. We believe that engineers should express themselves forcefully on this point, and even more forcefully in opposition to the notion that they are arrayed in hostile camps prepared to contend one party for wholesale construction of detention reservoirs and the other party for universal levee construction as the only proper method of flood control for the United States.

Designing Flexible Joints for Submerged Pipe Lines.

The subjects of design and construction of submerged water pipe lines are ones of perennial interest to water works engineers and water department officials. The editors frequently are reminded of this fact by the numerous requests received from readers in the water works field for information relative to the best methods of handling submerged pipe line work. It is, therefore, with the greatest confidence in its weight, that we call attention to the article entitled The Designing of Flexible Joints for Submerged Water Pipe Lines of Cast Iron and Steel, which we publish in the water works section of this issue.

In the article mentioned 29 forms of flexible joints are illustrated and described. The merits and defects of each form shown are critically discussed. Our contributor, Mr. Emil

Kuichling, has made a very thorough study of the literature of this subject and in his article reviews for our readers the features of all of the published descriptions of flexible joints. He also indicates the rational method of stress computation for the component parts of flexible, spherical joints. The essentials of strong and serviceable joints of this type are clearly presented in his discussion.

The article relates almost wholly to the design of joints and but incidentally to the actual laying of submerged lines. Little is said about the employment of divers for making up submerged joints, and of the precautions necessary to insure dependable joints made in this manner. In his long and extensive practice in water works engineering Mr. Kuichling has found it wise to place more dependence upon well-designed joints made up above water by competent workmen than upon joints put together with great difficulty by divers. His article is, therefore, restricted to a thorough study of the design of joints rather than to a study of construction methods employed in laying submerged pipe lines.

This article should be read and preserved for reference by every water works engineer. The author's treatment of his subject is so comprehensive and so clear that his readers will in future approach problems of this character with greater confidence in their ability to construct dependable submerged pipe lines containing flexible joints.

Four Months Allowed for an Appraisal of Public Utilities in Ohio.

The Public Utilities Commission of Ohio has issued an order requiring nearly all classes of utilities under its jurisdiction to file an appraisal of their properties before the first of August. The giving of only four months in which to perform so large a task—large at least for the large companies—indicates that the Commission does not appreciate the amount and kind of work involved in an appraisal of this sort. A year would be none too much time in which to appraise the property of a large company, and we doubt not that it will be necessary to extend the time at least eight months beyond the four that are specified.

In this connection it may be remarked that there is a growing tendency on the part of commissions to require utility companies to submit appraisals that are subsequently checked by the engineers of the commission. It is rare that engineers differ much as to the quantities listed in an appraisal, yet it is the gathering of the quantities that takes most of the time and causes most of the cost of an appraisal. Moreover, it is usually not difficult to check the quantities, provided they are reported in such a way as to facilitate checking. Thus if the count of poles is recorded by lines or by areas, it is a simple matter to recount certain lines or areas at random; and if the random recount checks the count there is small doubt as to the accuracy of the entire count.

Hurriedly made appraisals are apt to be a curse to all concerned. Not only is property overlooked, but, what is of much greater importance, unit costs are grossly underestimated. A careful study of cost involves a thorough analysis of accounting records. This fact is one that comparatively few would-be appraisal engineers even yet appreciate. The tendency is to spend most of the time on the inventory of quantities, and then hastily decide upon the unit prices to be applied. A proper study of the unit costs in a large city like Cleveland would keep a staff of expert engineers busy six or eight months. A unit-cost investigation is not the sort of work that can be split into many elements and assigned to many men. The elements are so interdependent and the importance of the work is so great that unit-cost studies must be delegated to a few very well trained engineers if there is to be justice done.

Another Non-Technical Board Recommended to Supervise Engineering Work.

As a result of an investigation of the state-aid road work in New York state, Mr. Jas. W. Osborne, a special commissioner of investigation appointed by the governor, has made an interesting and valuable report. Of course he finds that road work should be "taken out of politics," and quite naturally he recommends the creation of a commission whose function it shall be to select a chief engineer, "who shall be the responsible executive head of the [highway] department with a salary sufficient to compensate and attract an engineer of eminence."

All this is excellent, but all this excellence is marred by the further recommendation that the "board of highway commissioners shall consist of representative citizens to be selected one from each of the eight judicial districts of the state, who shall serve without salary."

The governor is to appoint this board of eight "representative citizens." Their condition of appointment, it seems to us, is fraught with political danger. There is doubtful merit in the suggestion that they be men of different political faiths, since this very difference among political appointees would probably result in friction. The aim evidently is to secure a board corresponding to a board of directors who shall select a chief engineer and have a voice in the larger policies of executive management. The means to this end seem to us inadequate.

Highway construction is almost entirely a technical matter. The executive in charge of it should unquestionably be an engineer. And by an extension of the same reasoning that leads to this conclusion, it must be also concluded that the highway commission should itself be mainly composed of engineers and road builders. We add "road builders," for among contractors there are many men who are well informed both about road economics and about the ability of engineers as economists and executives.

Certainly there is advantage in having a highway commission that has enough technical knowledge to be able to weigh intelligently the merits of engineers who are under consideration as the executive head of a highway department. Even such a highway commission as that suggested by Mr. Osborne is infinitely to be preferred to popular election as a means of securing a competent head of a highway department. Is it not clear, however, that as a commission of lawyers is best qualified to select a judge, so a commission of road engineers and contractors is best qualified to select a highway engineer?

Water Works

The Designing of Flexible Joints for Submerged Water Pipe Lines of Cast Iron and Steel.

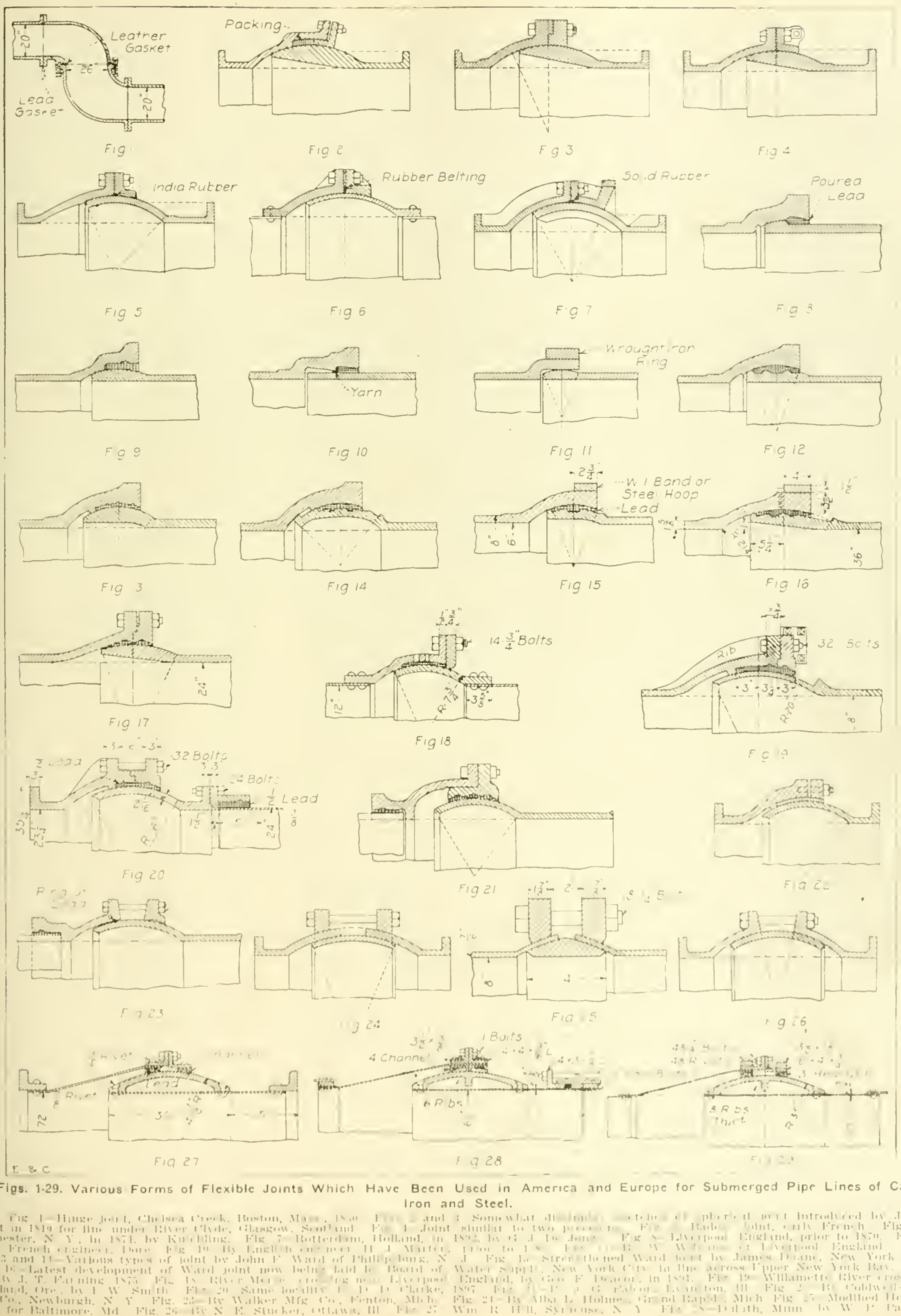
Contributed by Emil Kuichling, M. Am. Soc. C. E., Consulting Civil Engineer, New York City.

It sometimes happens in water works practice that a line of heavy iron pipe must be laid across a river, or other body of comparatively deep water, in cases where a suitable bridge is not available and the natural bed of the body of water is of more or less yielding material into which the heavy pipe may be expected to settle to some extent. In such a case the usual comparatively rigid joints become inadmissible unless a firm foundation for the pipe is provided and a number of joints are made by divers under water. Since both of these expedients may involve large expense and uncertainty, consideration in such cases is given to the use of flexible joints which can be coupled securely above water and which will be practically watertight against either internal or external pressure after all movements of the parts, due to lowering a line of several lengths of pipe and its subsequent settlement into the yielding bed, are completed. Very little information about the principles of proportioning the parts of such a flexible joint is given in most text-books on water works, or in the articles published in engineering journals describing the laying of submerged pipes, and therefore it has seemed to the writer that a more comprehensive treatment of this matter will be of interest.

The subject is very broad, and its consideration must here be limited to flexible joints of cast iron and steel pipes of relatively large diameter and subjected to much pressure, thereby excluding references to connections made with leather, rubber and canvas sleeves and interlinked sections of thin corrugated metal and wire. Flexibility may be provided in one or more directions or planes. The familiar swinging gas-light bracket permits motion in only one plane, but by using a combination of several such joints and links of pipe, the axes of some joints being at right angles to the axes of the others, the burner can be brought into almost any desired position. This form of joint is very old. It is shown in connection with the discharge pipe of hand fire-pumps in Belidor's "*Architecture Hydraulique*," Paris, 1739, and in Leupold's "*Theatri Machinarum Hydraulicarum*," Leipzig, 1774; and probably it was not a novelty at that time. The only instance of its use with a large submerged water main appears to have been in a line of 20-in. cast iron pipe laid in 1850 under Chelsea Creek at Boston, Mass., and which was taken up and replaced by a larger pipe about 1899, as described by C. M. Saville, C. E., in Engineering News of Feb. 28, 1901. A section of this joint is shown in Fig. 1.

These 20-in. pipes were cast in 9-ft. lengths with flanged ends, and the flexible joint was inserted at every third length, thus forming a series of offsets about 30 ft. long which are rigid laterally, but admit of motion vertically. The amount of such motion, however, is necessarily limited because the pipes are bolted together tightly, and any appreciable increment in the length of the line due to settling must be obtained by a corresponding slipping at the terminal socket joints. The hinge joints were made watertight with thin leather gaskets covered with white lead, and when the parts were disconnected after nearly 50 years, the leather was found to be in excellent condition. In this type of joint the residual tensile strength of the bolts in the flanges, after compressing the gasket, should be equal to that of the barrel of the pipe, and the clamp-ring at the hinge must be able to resist the longitudinal forces caused by the elongation or contraction of the flanged sections of pipe by changes of temperature. It would, however, be preferable to minimize this action by using one or two ordinary socket joints in each 30-ft. section.

Flexibility in all directions is afforded by a ball and socket or spherical joint without much more expense for labor and material than is involved in the above-described hinge joint. The spherical joint seems to have been invented by James Watt in 1810 for a 15-in. cast iron water pipe which was laid in that year under the River Clyde at Glasgow, Scotland, and reference to the invention is made in Arago's "Éloge Historique de Watt," read before the French Academy of Sciences on



Figs. 1-29. Various Forms of Flexible Joints Which Have Been Used in America and Europe for Submerged Pipe Lines of Cast Iron and Steel.

Fig. 1—Hinge joint, Chelsea Creek, Boston, Mass., [illegible]. Figs. 2 and 3—Somewhat dissimilar sketches of spherical joint introduced by James Watt in 1810 for line under River Clyde, Glasgow, Scotland. Fig. 4—Joint similar to two preceding. Fig. 5—[illegible] joint, early French. Fig. 6—Rochester, N. Y., in 1874, by Kuichling. Fig. 7—Rotterdam, Holland, in 1892, by G. J. De Jong. Fig. 8—Liverpool, England, prior to 1870. Fig. 9—By French engineer, Dore. Fig. 10—By English engineer H. J. Marten, prior to [illegible]. Fig. 11—E. W. [illegible] of Liverpool, England. Figs. 12, 13 and 14—Various types of joint by John F. Ward of Phillipsburg, N. J. Fig. 15—Strengthened Ward joint by James Duane, New York City. Fig. 16—Latest development of Ward joint now being laid by Board of Water Supply, New York City, in line across Upper New York Bay. Fig. 17—By J. T. Fanning, 1875. Fig. 18—River Mersey crossing near Liverpool, England, by Geo. F. Deacon, in 1891. Fig. 19—Willamette River crossing, Portland, Ore., by I. W. Smith. Fig. 20—Same locality, by D. D. Clarke, 1897. Fig. 21—[illegible], Evanston, Ill. Fig. 22—By Coldwell-Wilcox Co., Newburgh, N. Y. Fig. 23—By Walker Mfg. Co., Fenton, Mich. Fig. 24—By Alba L. Holmes, Grand Rapids, Mich. Fig. 25—Modified Holmes joint for Baltimore, Md. Fig. 26—By N. E. Stucker, Ottawa, Ill. Fig. 27—Wm. R. Hill, Syracuse, N. Y. Fig. 28—Duluth, Minn., by W. F. Patton. Fig. 29—Rochester, N. Y., by E. Kuichling.

Dec. 8, 1834, in the following picturesque terms:

> Watt was consulted about the serious problem of laying a large water pipe across the river, and his solution was soon ready. A few days previously he had seen a lobster on the table and had discovered how the mechanism of the crustacean's tail might be reproduced in iron so as to form a series of articulations which would have complete flexibility. He therefore suggested the use of an articulated iron pipe which would be able to adapt itself to all present and future irregularities in the bed of the river. The Glasgow Water Co. thereupon ordered such a pipe 1,000 ft. long to be made and laid in accordance with Watt's plans, and its success was complete.

The invention is also mentioned briefly in Burnet's "History of the Water Supply to Glasgow," published in 1869.

The writer has made an extensive search in engineering literature for an authentic drawing of Watt's ball-joint, and was able to find only two somewhat dissimilar sketches of it. One of these is given by Prof. L. J. Le Conte, C. E., of the University of California, in a discussion of flexible joints in Trans. Am. Soc. C. E., Vol. 33 (1895), p. 284, and the other is contained in G. Meissner's "Die Hydraulik," Vol. 1. p. 553, Jena, 1878. The latter is like Fig. 3, but as this design has been attributed to some other British engineer by different authors, the writer gives preference to Prof. Le Conte's drawing, which is here reproduced as Fig. 2. It is described as follows:

> The 15-in. cast iron pipe was made in 9 ft. lengths with flanged ends, and had ball-joints interposed at variable distances, depending upon the profile of the bottom of the trench. The length of the sections between ball-joints was generally 28.5 ft., except at the middle of the line, where a single 10 ft. length became necessary in order to fit the ground. The ball surfaces were not turned at all, being made watertight by means of an adjustable packing joint similar to an ordinary stuffing box.

It should be remarked that the flanges at the outer ends of the ball and bell castings in Fig. 2 were added by the writer to conform to a small drawing of the entire river-crossing printed in Smiles' "Lives of Boulton and Watt," London, 1866, which indicates that such flanges were used.

A brief verbal description of Watt's ball-joint is also given in John M. Goodell's "Manual of Water Works for Small Cities and Towns," New York, 1899, p. 209. It is there stated that the ball "was held within the bell by a lead ring pressed firmly into place by a ring of angle-iron bolted to the bell." From Fig. 2 it appears that this clamp-ring was a single casting having a diameter large enough to slip over the end flange of the ball piece, but smaller than the diameter of the ball, otherwise it would have to be divided into two halves; and it is also plain that the spherical flange of the ring could not be inserted snugly into the bell unless the inner surface of the latter is cylindrical or slightly conical. Furthermore if the spherical surfaces of the ball, bell and ring were left rough, instead of being accurately turned in a lathe, considerable leakage would have resulted from the abrasion of the lead packing by the repeated movement of the rough ball in the process of laying the pipe, and the work could not be called completely successful; but with an elastic fibre packing, such as is used in stuffing boxes, and a fairly smooth surface of the ball, the joints might have remained practically watertight. The parts, however, were not adapted to resisting much longitudinal tension, and hence the sections of pipe were fastened to timber platforms of corresponding length provided with strong wrought iron hinges at each of the ball joints, whereby the pipes were relieved of stress while being deposited in the trench.

After Watt's invention became more widely known, several modifications of the ball and socket joint were made by European engineers. These were characterized by accurate workmanship and the omission of a lead bearing. Figure 3 is taken from Becker's "Wasserbau," Stuttgart, 1856, and the descriptive text attributes its design and use to English engineers whose names are not mentioned.

Figure 4 is a similar design taken from Humber's "Treatise on the Water Supply of Cities and Towns," London, 1876, and Chicago, 1879. In this design, and probably also in the preceding one, the clamp-ring is made in two halves, so that the tendency to burst it by a pull on the joint is transmitted to the flange of the bell. In this respect preference must be given to Fig. 3, unless the bolts indicated in Fig. 4 fit tightly into the bolt holes. Watertightness is ultimately obtained by rusting or the accumulation of fine silt.

An early French design is Badois' joint, Fig. 5. It is characterized by cutting away the heels of the diametral flanges to form a triangular groove, into which a thick cord of India rubber is placed and powerfully compressed by the flange bolts, thus securing initial watertightness. This joint is shown in nearly all French treatises on water supply published since 1860. It will be noticed that the clamp-ring is here continuous, and that the diameter of the ball is made large enough to allow the ring to pass over the end flanges. The joint is also made with ordinary spigot ends for use with connecting pipes having the common sockets.

In regard to the sizes of these early joints, little information is available except that after Watt's death in 1819, his spherical joints were used in three other lines of water pipe under the Clyde at Glasgow, namely, an 18-in. line in 1830, a 25-in. line in 1838, and a 40-in. line in 1840; also that Badois' joint was used in a French river crossing with 12-in. pipe.

In 1871 a joint similar to that of Badois was designed by the writer for use with a deflection of 20.7° on the long 36-in. wrought iron intake pipe of the first conduit of the Rochester, N. Y., water works. This joint is shown in Fig. 6. The tendency to burst the clamp-ring by a pull on the joint was recognized by strongly ribbing the diametral flanges, and watertightness was secured by a closely fitting narrow ring of rubber belting, which was slipped over the ball between the flanges and compressed by the bolts until stopped by thin sheet iron rings or washers on the bolts. These joints were used very successfully on the work, and subsequently similar joints of smaller sizes were designed for the water intakes of two or three other municipalities. An improvement would be made by substituting for the separate washers a continuous ring of plate iron perforated for the flange bolts.

Another modification of this type of joint is the one designed in 1892 by G. J. DeJough, C. E., for a 24-in. submerged pipe at Rotterdam, Holland, and published in the Transactions of the Dutch Society of Engineers for 1893. It is shown in Fig. 7, and is characterized by a large overlap of the clamp-ring, the reinforcement of the latter with a thick steel ring shrunk upon the radial flange, and the strengthening of the ball, bell and flanges by means of ribs. The dimensions are such as permit the clamp-ring to pass over the end flanges and allow a deflection of 26°. Watertightness is secured by means of a ring of solid rubber compressed in the triangular groove formed at the heel of the diametral flanges. The joints weighed 5,075 lbs. and cost $159.20 each, complete.

It may be remarked that 'the early cast iron pipes were made with flanged ends, bolted together and consequently very rigid. Numerous breaks occurred in the mains from settlement and contraction in cold weather, so that a demand soon arose for a different and less expensive style of connection. Various expedients were tried, such as screw joints and plain spigot ends in smooth sleeves with wooden packing, etc., until at some time between the years 1780 and 1820 pipes were made by casting a simple bell or socket at one end. They were connected by placing a gasket around the spigot end and filling the remainder of the annular space in the socket with melted lead, substantially as done at the present time.

In a discussion on water works in Proc. Inst. C. E., Vol. 14, 1855, p. 40, the distinguished English engineer James Simpson stated that, according to the evidence before a committee of the House of Commons in 1821, "the socket joint originated in a broad iron collar being placed over the ends of two pipes whose flanges had been cut off, the cavity being subsequently filled with lead and set up. Prior to 1746 socket pipes had scarcely ever been used." These joints afforded the requisite slight flexibility to facilitate laying in a trench and prevent fracture by settlement, and also allowed each length of pipe to contract in winter by slipping slightly on the lead packing. After the reliability of lead as a packing material had been established by long experience, it was applied to ball and socket joints in order to obtain watertightness and reduce the expense of lathe work by turning only one of the spherical surfaces and allowing the melted lead to form the corresponding surface. This process developed a second type of flexible joint, of which the following are representatives:

To provide only a slight flexibility, such as occurs from the settlement of heavy pipes in yielding soil, an English engineer, whose name is unknown to the writer, designed at some time prior to 1870 the joint shown in Fig. 8, which is taken from Plate 12 of Humber's "Treatise on the Water Supply of Cities and Towns," London, 1876. A short length of the bell and spigot was machined to a true spherical surface, and the joint finished by a poured lead packing recessed into the bell and calked. This device was used successfully on lines of large mains in Liverpool. A similar joint, designed by the French engineer Doré, at a date unknown to the writer, is shown in Fig. 9. This illustration is taken from Lueger's "Wasserwersorgung der Stadte," Leipzig, 1908, Vol. 2, p. 107, where it is described as one of the oldest forms of flexible joint. It resembles closely Ward's joint, which is described below.

Another method of securing moderate flexibility in a socket joint is shown in Fig. 10. It was devised by the English engineer, H. J. Marten, and is described by him in Proc. Inst. C. E., Vol. 74, 1883, p. 176, where he states that a 22-in. main which he had provided with such sockets some years before had settled without leakage a vertical distance of 13 ft. in a length of 300 to 450 ft. owing to a subsidence of the ground caused by mining operations. The frequency of subsidences in the mining districts of England stimulated engineers to invent pipe joints that would yield and still remain water tight. One of the most interesting of these devices is that of W. Williams, of Liverpool, shown in Fig. 11, taken from London "Engineering" of July 9, 1869. The spigot of the pipe is provided with a short spherical surface whose diameter is slightly greater than that of the cylindrical socket, both surfaces being carefully turned and bored. The socket or bell is also strongly reinforced by having a thick ring of wrought iron shrunk upon it. Before inserting the spigot end, the bell is expanded by heating, and after cooling it will grip the spigot powerfully. The tests of this joint exhibit remarkable results, but it did not come into extensive use.

The best-known spherical joint for water pipes is that which was invented in 1862 by the late John F. Ward, M. Am. Soc. C. E., while superintendent of the Warren Foundry & Machine Co., of Phillipsburg, N. J. In that year he laid a long line of 8-in. pipe fitted with this joint across the Delaware River from Easton, Pa., to Phillipsburg, and in the following year he obtained a U. S. patent (No. 39,691) on his invention, which was termed an "Improvement in Pipe Coupling." The device is made in several ways, as shown in Figs. 12, 13 and 14, and has been used for many lines of submerged pipe of all sizes from 6 to 36 ins. in diameter.

In Fig. 12 the bell is bored to a true spherical surface for the required overlap on each side of the central diameter, and the cylindrical spigot is thickened and grooved, the end being turned in a lathe. The annular space between the bell and spigot is then filled with melted lead, which is held to the spigot by one or more grooves and slides over the smooth spherical surface of the bell when the spigot is deflected in any direction; and if a slight leakage develops on testing, the end of the lead at the face of the bell can be calked. Objection is sometimes made to the retreat of the edge of the lead within the bell on one side, and its projection beyond the bell on the opposite side, whereby calking is rendered difficult; hence the spigot is then provided with a smoothly turned spherical surface, while the interior of the bell is cast with grooves to hold the lead immovably, and a projecting ring at the inner end of the lead is bored to match the smooth surface of the spigot, as shown in Fig. 13. In this style of joint the outer edge of the lead packing is always flush with the face of the bell, and can therefore be readily calked.

In some cases it was deemed desirable to limit the degree of flexure and bring the parts to a firm bearing, without injury to the lead packing or danger of splitting off the inner edge of the bell. This was accomplished by modifying Fig. 12 so as to provide two stop rings on the spigot and a shoulder in the bell as indicated in Fig. 14.

Experience with these spherical joints showed that there was a tendency to burst the cast iron bells when a strong longitudinal pull was developed in the line, and this led to the practice of reinforcing the bell with a thick hoop of wrought iron or steel shrunk upon it, as shown in Fig. 15. This method of strengthening Ward joints was applied in 1888 by the late James Duane, M. Am. Soc. C. E., then assistant engineer in the Water Department of the city of New York, as described in Engineering News of May 18, 1889, and has been continued very successfully with all the submerged pipes laid by the city since that time. These pipes range in size from 6 to 36 ins., and an excellent account of the manner in which they were laid, tested and repaired is given in a paper by W. D. Lintz, C. E., published in the "Proceedings of the Municipal Engineers of the City of New York," 1904, pp. 130-155. The latest development of this joint, Fig. 16, is now (1911) being carried out by the New York Board of Water Supply in a line of 36-in. cast iron pipe about 9,800 ft. long, laid in a dredged trench in the irregular bottom of Upper New York Bay from Bay Ridge in Brooklyn to Tompkinsville, Staten Island.

The joint is formed on the ends of each pipe so as to make the deflection about 10°-30', the laying length 12 ft. and the weight 8,950 lbs.; a wrought iron or steel tire 4 ins. wide and 1½ ins. thick is to be pressed or shrunk upon the smoothly turned cast iron bell, which has a face thickness of about 3½ ins., and whose inner surface will be bored to a true spherical surface having a radius of of 21¼ ins.; the melted lead packing has a width of 8½ ins., a thickness ranging from 7/16 to ¾ in., and a weight of about 280 lbs.; and a novelty is the provision in each bell of 16 equidistant steel gib screws, ¾ in. in diameter with squared end. After the lead has been poured, these screws are removed and a stiff mixture of graphite and thick mineral grease is forced into the shrinkage space between the lead and the bell; then soft lead plugs ½ in. in diameter and 1 ins. long are placed in the holes and compressed by means of the gib screws sufficiently to make a joint which shall remain watertight after being deflected 10° in any direction.

It is evidently expected that by thus forcing a little more lead into the packing the latter will become compressed and be made to flow laterally and fill any cavity or space under the bell; but this process is associated with the danger that the plugs may become hardened by the powerful pressure of the gib screws and score the softer spherical surface of the cast iron when movement takes place, thereby causing leakage; the latter, however, can be stopped by having a diver calk the exposed edge of the lead packing at the face of the bell. The behavior of lead under such conditions is still obscure and deserves careful investigation. From a number of experiments made by the writer about 20 years ago, it was found that soft lead would yield or flow slightly if subjected for a long time to much lower pressures than are usually given in handbooks for the ultimate compressive strength of the metal. The latter is frequently said to range from 1,600 to 2,000 lbs. per square inch, depending on the ratio of the height of the test specimen to its base; and in his "Engineer's Pocket-Book," 1902, p. 921, Trautwine states that a prism of soft lead 1 in. square in cross-section and 2 ins. high suffered a reduction of 1/200 its height under a pressure of 1,200 lbs., while the writer found a slight yet appreciable compression in a sample about ½ in. thick after constant pressure of about 1,000 lbs. per square inch during two weeks.

A flexible joint with lead packing was designed in 1875 by the late J. T. Fanning, M. Am. Soc. C. E., for a 24-in. submerged pipe, which is shown in Fig. 17 taken from p. 463 of a recent edition of his book on "Hydraulic and Water Supply Engineering," New York, 1899, where it is described as a modification of the Glasgow universal joint, as follows:

> The difficulty heretofore found in making the back part of the lead packing firm and solid is here overcome by separating the bell into two parts, so as to permit both the front and rear portions of the packing to be driven. The loose clamp-ring is pushed back and the ball entered into the bell nearly to contact; the lead packing is then poured and tightly driven, after which the clamp-ring is bolted into position and its lead packing poured and firmly driven. This secures solid lead in front and rear.

The joint shown permits a deflection of 11°-15', and the clamp-ring is continuous and large enough to slip over the ball.

In 1891 the late Geo. F. Deacon, M. Inst. C. E., found it expedient to lay a line of 12-in. lap-welded steel pipe 800 ft. long on the irregular bed of the River Mersey, near Liverpool, England, to afford a temporary water supply, and used the spherical cast iron joint with lead packing shown in Fig. 18. The description is taken from London "Engineer" of June 5, 1891. The exterior of the ball and the rear interior portion of the bell were turned to a true spherical surface, and the front portion of the bell was provided with two ¾-in. holes, 1 in. apart, through which to pour the melted lead, and also with a ⅜-in. hole to serve as a vent for gases formed in the space during the operation.

It will be noticed that the castings are very light, and yet they withstood severe strains while the pipe was being pushed across the river in 27½ minutes by pulling on wire ropes attached to the rear end. The entire line of pipe had previously been coupled together on greased ways on one shore, and the ends were closed tightly by plugs; and as the pipe moved outward its head end was supported from a boat, two other boats being brought into similar service at intervals of about 250 ft. After it had been pulled over, the plugs were removed and water admitted, whereupon its sinking to the bottom was controlled from the two intermediate boats.

A modification of Fanning's joint was made in 1893 by J. W. Smith, Chief Engineer of the Portland, Ore., Water Board, for a line of 28-in. cast iron pipe about 2,000 ft. long to be laid under the Willamette River in that city. The design is shown in Fig. 19, which is taken from a very interesting account of the work by Messrs. F. and A. S. Riffle, in Trans. Am. Soc. C. E., 1895, Vol. 33, p. 257. Each pipe had the unusual length of 17 ft. and weighed very nearly 5 tons. On being coupled together the length between centers of balls was 16 ft., and provision was made for a deflection of 15°. The large overlap of 6¾ ins. on the ball, whose exterior diameter was 40 ins., caused the smallest diameter of the clamp-ring to become about 39¼ ins., or less than the diameter of the ball, so that it was necessary to make it in two halves with flanges for bolting together, and to transmit the bursting force developed by a pull on the joint to the continuous flange of the bell, the split clamp-ring was recessed into the bell flange, as shown. The bell was also strengthened longitudinally by 32 equidistant ribs. Machine work was required on the exterior spherical surface of the ball, the faces of the clamp-ring and bell, and the projecting rib on the interior of the latter where the lead packing is stopped and a bearing is given to the ball. The lead was first poured and calked into the space between the ball and the bell; then the clamp-ring was bolted in place; and, lastly, the space between the ring and the ball was filled with melted lead, the calking of which caused the split ring to bear against the shoulder of the recess in the bell-flange.

It happened that the lathe-work on the balls did not produce truly spherical surfaces. This caused much fear that the watertightness of the joints would be injured seriously, but as the time necessary for correcting the error was not available, a test was made of a line of eight pipes that had been connected and laid in place. The outer end was closed by cast iron cap and water was admitted from the main on the shore, the expectation being that the hydraulic pressure on the terminal cap would produce a longitudinal pull on the joints sufficient to cause the lead to flow and stop any leakage. At 10 lbs. pressure per square inch it was found that every joint leaked considerably, but on increasing the pressure the leakage diminished, until at 45 lbs. it ceased and the eight leaky joints had become entirely tight. The success of this trial justified the acceptance of the slightly defective lathe-work, and the laying of the pipe was allowed to proceed on condition that each additional length of 200 ft. be tested in like manner. When a length of about 650 ft. of pipe had been laid, a pressure of nearly 250 lbs. per square inch was applied for a few minutes, which had the effect of pulling the spigot end of the shore pipe about ¼ in. out from its socket. In all 13 tests were made, the last one requiring the maintenance of a water pressure of 200 lbs. per square inch for 24 hours. The leakage at this time was estimated as being about ¾ gal. per minute. "During each of the 11th and 12th tests a bell burst, the fracture extending spirally around the pipe for half its length, and probably being caused by the wedging of the ball in its socket."

From these data a computation can be made in regard to the longitudinal pull exerted on the joints. With a pressure of 45 lbs. per square inch on the cross-sectional area of the 28-in. pipe, the pull was $= 615.75 \times 45 = 27,709$ lbs.; with 200 lbs. pressure it was 123,150 lbs., and with 250 lbs. pressure it was 153,937 lbs. It is also stated that the leakage stopped when the smallest of these three pulls acted in the first test. This implies that the lead packing had yielded somewhat to the radial pressure of the ball against the clamp-ring caused by the pull of 27,709 lbs. The bearing surface of the lead under the clamp-ring is about 3 ins. wide upon the ball, and the angle made by the middle radius of this surface with the vertical axis of the ball is $\alpha = 13° - 17°$; hence the vertical diameter of the ball at this point is [illegible] = 38.85 ins., the mean circumference is 122.05 ins., and the area of said surface is about 366 sq. ins. The average radial pressure q in pounds per square inch against this surface is balanced by the corresponding portion p of the said longitudinal pull $P = 27,709$ lbs., and as $p = \frac{27,709}{366} = 75.7$ lbs., we have from the parallelogram of forces described below

$$q = [illegible] \left([illegible] \right) \left(\frac{75.7}{[illegible]} \right) = [illegible] \text{ lbs.}$$

for the average stress on the said portion of the lead packing. This stress, however, depends on the angle α, and if it were computed for the adjoining equal area under the flange of the bell, we would have $\alpha = 14° - 18°$, and an average stress of

$$q = \frac{75.7}{0.08136} = 930.4 \text{ lbs. per square inch.}$$

With the much higher water pressures used in the tests, it is not surprising that the joints became tight and that some of the bells burst.

In 1897 plans were made for another submerged pipe under the Willamette River at Portland, Ore. This pipe was to consist of lap-welded steel tubes ⅜-in. thick, with an exterior diameter of 24 ins. and from 18 to 20 ft. long. The separate lengths were to be connected by means of cast iron flanges, and a flexible joint permitting a deflection of 15° was to be inserted at every second length. This joint, which was designed by D. D. Clarke, M. Am. Soc. C. E., now chief engineer of the Portland water works, is shown in Fig. 20, taken from Engineering News of Feb. 10, 1898. The specifications required the spherical joints to be put together in the shop and tested under a water pressure of 250 lbs. per square inch. The smoothly turned ball is inserted until it rests on the internal projection of the bell, which is bored to fit; a thin gasket is then driven against said projection and melted lead is poured into the remainder of the annular space. The ball is then removed to ascertain whether the lead casting is perfect, and if not, it is cut out and the process repeated. The ball is then replaced and the continuous clamp-ring is applied with a separating gasket in the recessed joint; another thin gasket is placed over the lead and the annular space between the ball and the clamp-ring is filled with melted lead; the clamp-ring is then withdrawn and the lead casting examined. If perfect, the separating gasket in the recessed joint is removed and the clamp-ring bolted up securely. In case of leakage the exposed edge of the lead packing is calked, and while the hydraulic pressure is acting, the ball is to be turned in every direction to prove the workmanship. Provision is also made for shrinking thick steel hoops around the flange of the bell and clamp-ring.

It was doubtless intended to retain the lead packing within the compound socket formed by the bell and clamp-ring, and presumably the interior surfaces of these members were to be made rough enough for the purpose. Such roughness, however, is not indicated in the illustration referred to and hence the lead-locking recesses in bell and clamp-ring, shown in Fig. 20, were added by the writer. The separation of the packing by a gasket at or near the recessed joint, affords opportunity for leakage through the same, but this can be avoided by using a thin gasket in said recess. In this design the radial stress against the clamp-ring, due to a longitudinal pull, is not transmitted to the bell, but the overlap at the diametral joint will carry to the bell most of the force required to turn the ball in its lead bearings. It is also of interest to note the connection of the steel pipes to their cast iron end flanges by means of lead packing, as with the usual bell and spigot pipe. With such connections the application of high water pressure for testing sections of the submerged pipes does not seem very desirable; but on the other hand, these socket joints will allow each length of pipe to contract by reduction of temperature without inducing a great longitudinal pull on the ball joints. The design, therefore, gives evidence of very thorough study.

The magnitude of the longitudinal force or pull exerted by the contraction of a wrought iron or mild steel pipe due to lowering its temperature is easily computed. For each degree Fahrenheit, the elongation or contraction of these two metals is about 0.0000056 of their length, and their modulus of elasticity in tension or compression is about 30,000,000 lbs. per square inch. The range of temperature in natural streams and public water supplies is about 45° F., or from 77° in summer to 32° in winter. Hence the contractile stress in winter upon the metal of a pipe that was laid in summer between two rigidly fixed points will be $= 45 \times 0.0000056 \times 30,000,000 = 7,560$ lbs. per square inch of sectional area. If, however, the temperature of the submerged pipe at the time of laying was 62° instead of 77° F., the contractile stress in winter will be only $= 30 \times 0.0000056 \times 30,000,000 = 5,040$ lbs. per square inch. In the case of the 24-in. steel pipe mentioned above, the sectional area of the metal is 27.83 sq. ins., so that if the contractile stress were only 5,000 lbs. per square inch, the longitudinal force or pull on the ball joints would be 139,150 lbs. unless the intermediate socket joints yield somewhat. An axial pull of this magnitude corresponds to a water pressure of 328 lbs. per square inch against the capped end of the said pipe, and for the given overlap of 6 ins. the ball will exert an average compressive stress of 1,252 lbs. per square inch against the lead packing under the clamp-ring.

Another form of spherical joint with lead packing is shown in Fig. 21. This was invented by Jos. G. Falcon of Evanston, Ill., and is described in Engineering News of June 8, 1893. It requires careful machine work on the surface of the ball, and also on the flanges of the bell and clamp-ring to make a close fit in order to avoid fracture of the latter. The joint has been made for sizes up to 24 ins., and has been used successfully on numerous pieces of submerged pipe work.

The flexible joint shown in Fig. 22 is made by the Coldwell-Wilcox Co., of Newburgh, N. Y., for both steam and water pipes. The clamp-ring is here fastened to the bell with tap-bolts, which must be strong enough to prevent bending when the pressure due to turning the ball is brought into action, and for the same reason these bolts should fit snugly into the bolt holes of the ring. Care should also be taken to secure ample overlap of the clamp-ring in order to avoid excessive stress on the lead packing.

The ball-joint shown in Fig. 23 is made by the Walker Manufacturing Co. of Fenton, Mich., in sizes up to 24-in. In this design a generous overlap is provided, but a powerful leverage is thereby exerted on the bolts when the ball is turned. At the bell watertightness is sought by the compression of an inlaid hoop or ring of lead, while at the clamp-ring a suitable bearing alone is required. If there is much movement of the ball, after the joint is made, the narrow lead packing in the bell may become abraded or unduly compressed in places, whereby leakage will ensue; hence some provision should be made for conveniently stopping such leakage otherwise than by driving wedges under the flange of the bell.

A somewhat similar joint, invented by the late A. L. Holmes of Grand Rapids, Mich., is shown in Fig. 24, taken from Engineering News of Nov. 22, 1890. It consists of a ball, two bells and two clamp-rings with their necessary bolts. Careful lathe work is obviously required on all the spherical surfaces in contact, and means must be provided for holding the ball so that the planes of its edges will be parallel to the planes of the clamp-rings, at least while the joint is being made. In the description mentioned, reference is made to the use of a slightly projecting copper ring placed in a groove near each end of the ball to insure watertightness; but if such packing is omitted, dependence in this respect must be placed on bringing the bells into absolute contact with the ball by means of the clamp-rings and bolts.

A slight modification of this device was recently made in securing moderate flexibility for the joints of the new steel high-pressure water mains of Baltimore, Md. These pipes are of cast steel in lengths of 20 ft., and from 8 ins. to 20 ins. in diameter. The joint for 8-in. pipes is shown in Fig. 25, taken from Engineering Record of Sept. 10, 1910, and The Cornell Civil Engineer of 1910, p. 211. All the spherical surfaces are accurately turned in a lathe. While a joint is being made the steel segmental ball is held erect by means of a ring bolt screwed into a threaded hole at the top, until its edges are parallel to the two steel clamp-rings, and the clamp-bolts are then drawn up to bring the bells in contact with the ball. Tests under high water pressure proved that these joints did not leak and still retained some mobility. In general there is little fear of corrosion when steel or iron surfaces remain in contact, but if a portion of the surface is accessible to moisture, rusting will ensue and the frictional resistance to motion will be correspondingly increased.

Another style of spherical joint is shown in Fig. 26. It is flexible only while being adjusted to the required deflection, after which it remains rigid. The bolts are of different lengths, and beveled washers are needed to provide a proper bearing for head and nut unless they are suitably bent. In some cases watertightness is secured by inserting and compressing a narrow ring of lead or copper in a groove cut in each bell at the bottom of the flange, as indicated in Fig. 23. Joints of this kind were used in laying a line of 14-in. cast iron pipe 550 ft. long under the Illinois River at Ottawa, Ill. The work was done under the direction of City Engineer N. E. Stucker, and is described in Engineering News of April 18, 1895. It seems that wedge-shaped wooden gaskets were used between the flanges of the ball joints, and plain sheet rubber gaskets at the end flanges; also that the ball was kept from turning independently of the bells by a hinged bolt on each side. All bolts were inserted and drawn up by divers.

In the foregoing a variety of flexible joints adapted to pipes of diameters up to 36 ins., have been described, and it remains to refer to those of larger size. The great weight and cost of cast iron pipes of large diameter led, in 1888, to the use of comparatively light and cheap steel plate pipes and flexible joints of structural steel for the long intake of the Toronto, Ont., water works. These pipes were 48 and 60 ins. in diameter and were riveted up into sections 55 and 57 ft. long, respectively, the total lengths being 4,660 and 6,027 ft. To give them some opportunity to settle in the soft clay bottom of Toronto Bay in which they were laid, 65 flexible joints were inserted in the 48-in. pipe, and 15 in the 60-in. pipe. The joints were designed and made in 1888 by Wm. H. Law, C. E., of the Central Bridge and Engineering Co., of Peterborough, Ont., and in 1893 a number of similar 72-in. joints were made by him for the same city. A general description of this work is printed in Engineering News of Oct. 17, 1891, and June 29, 1893, as well as in the annual reports of the City Engineer of Toronto for the period of years indicated. It may also be remarked that Mr. Law's plan has subsequently been copied with slight modifications by other engineers.

Figure 27 shows the design for joints of the 72-in. pipe which permits a deflection of 15°. The ball is a hollow spherical zone of cast iron, 1¾ ins. thick after turning in the lathe, and is attached by many countersunk ⅞-in. rivets to a short cylindrical section of pipe ½ in. thick provided at its end with a steel 4-in. x 4-in. x 1-in. flange. The conical bell is made of the same steel plate, slightly bent over near the ends to bring it to cylindrical form to receive the end flange and a welded outside ring of steel angle 6 ins. x 3½ ins. x ⅝ ins., as well as an inside ring of steel channel 3 ins. x 1¼ ins. x ⅜ ins., whose edges are planed off to match the curve of the ball. The angle, plate and channel are securely fastened together with two rows of ¾-in. rivets, and after being fitted to the ball, the space within the channel ring is filled with melted lead entering through a narrow crevice between the ball and the edge of the channel. The clamp-ring is made in like manner, but the steel angle is here 6 ins. x 4 ins. x ⅝ in. to compensate for the plate thickness of the bell. The vertical legs of the angle rings are provided with holes for 1-in. bolts spaced about 6 ins. apart, and the joint is completed by placing a thin packing of soft dry wood between the two angle-rings and drawing up the bolts tightly.

In this joint the lead bearings on the ball have an overlap of 6 ins., which makes the versed sine of the arc covered by the bearings = 0.151 in.; but in the ball joints of the 48-in. pipe the overlaps and angles are only 4 ins., which gives a versed sine of about 0.28 in. While the latter may be large enough to prevent the joint from being pulled apart by the longitudinal contractile force in the sections of pipe, yet the resulting stress on the lead packing will be great enough to cause a considerable permanent compression and flow of the metal, with slight subsequent leakage when the temperature rises. The thickness of the pipe is here ⅜ in. and the sectional area of the steel is 56.9 sq. ins., so that with a contractile force of only 5,000 lbs. per square inch, the longitudinal pull will be 284,500 lbs.; the area of one lead ring 3 ins. wide is 530.4 sq. ins., and the angle from the vertical axis to the middle point of the bearing is 5° 04.6′; hence the average compressive stress on the lead is 6,061 lbs. per square inch, which will probably cause the lead to flow. Similarly, in the 72-in. pipe with a thickness of ½ in., the contractile force is 569,500 lbs.; the area of one lead ring 3 ins. wide is 752.2 sq. ins., and the angle from the vertical axis to the middle point of the bearing is 6° 26′; hence the average compressive stress on the lead is 6,757 lbs. per square inch, which is also excessive. This computation shows that the lead rings of both joints should have been much wider to conserve watertightness.

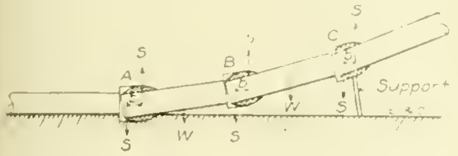


Fig. 30. Notation Diagram for Stress Computation in Unsupported Flexible Spherical Joint.

Flexible joints of this description were used in the long steel 54-in. intake pipe of the Syracuse, N. Y., water works, and the similar 60-in. intake pipes of the Rochester, N. Y., and Duluth, Minn., water works. The first of these works was carried out under the direction of Wm. R. Hill, M. Am. Soc. C. E., and is described in Engineering News of June 15, 1893, and also in Trans. Am. Soc. C. E., Vol. 34, 1895, p. 23; the second was done under the writer's direction in 1893-4, and is described in the Report of the Executive Board of the City of Rochester, N. Y., for 1894-5, Engineering News of April 11, 1895, and Trans. Am. Soc. C. E., Vol. 33, 1895, p. 273; and the third was planned in 1896 by W. P. Patton, M. Am. Soc. C. E., then City Engineer of Duluth, and is described in Engineering News of May 5, 1898.

In the Syracuse work it was decided to connect the several sections of the 54-in. intake pipe under water by means of divers, and one end of each section and flexible joint was accordingly provided with a strong cast iron cylindrical socket having an external flange near its middle to afford a bearing for a number of hook-bolts, by which a loose ring of ½-in. lead pipe placed around the spigot of the connecting section could be forced and expanded against the socket and a fillet riveted around the spigot end so as to make a tight joint. With such connections and the elasticity of the numerous hook-bolts, considerable room was given for the slight contraction of the sections in winter without causing a great longitudinal pull on the flexible joints, and hence it was deemed sufficient to equip the latter with diametral ring made of 4-in. x 4-in. x 9/16-in. steel angles and 4-in. channels whose flanges were planed off sufficiently to leave room for a melted lead packing ¾ in. thick.

The 60-in. Duluth intake pipe was laid in the same manner as the Syracuse pipe; near the shore it was placed in a deep trench 844 ft. long and extended 700 ft. farther out into Lake Superior to a point where the water was 67 ft. deep. Observations of the temperature variations of the water showed a very small range, the highest in summer being only 40° F.; hence in this case also the longitudinal forces acting on the ball-joints were inconsiderable. The diametral steel angles and channels were of the same dimensions as at Syracuse, but they were strengthened by adding rings of steel plate ⅜ in. thick, as shown in Fig. 28, which also shows the steel socket joints with hook-bolts and loose ring of lead pipe as packing. These latter joints were made by a diver, as at Syracuse.

In the Rochester work the 60-in. steel intake pipe was laid in water from 20 to 50 ft. deep without divers. The pipes were riveted together to form lengths of about 110 ft., to which the bell and ball parts, respectively, of the flexible joints were riveted, and were then transported as required on small pontoons to clusters of braced piles driven in the line at the points where ball-joints occurred. Two strong winches were located on each cluster of piles, one holding above water the bell end of the pipe previously laid, and the other supporting the ball end of the new pipe to be connected. When thus held by the winches the pontoons were free to return to the shore for another pipe, and meanwhile the diametral flanges were securely bolted together above water. The joint thus completed was then lowered to the bottom by one of the winches, water being admitted from the bell end of the new pipe, and the two winches were taken to the next cluster of piles. Where the bottom was soft, a large timber platform was attached below the flexible joint and sunk with it to serve as a support.

This joint, which permits a deflection of 17°-20′ in any direction, is shown in Fig. 29. It is characterized by a large overlap of the 3-in. lead bearings, resulting from the use of welded 6-in. x 4-in. x ¾-in. steel angles for the diametral flanges, and the substantial reinforcement of the same by welded steel rings 3½ ins. wide and 1½ ins. thick. The diameter of the ring on the clamp bearing is 73 ins., and with a stress of 15,000 lbs. per square inch it will resist a bursting pressure of 616 lbs. per square inch; the average diameter of the lead bearing is 69.42 ins., corresponding to a circumference of 218.1 ins., and as this bearing is 3 ins. wide, its area is 654.3 sq. ins.; the angle that the middle of the bearing surface makes with the vertical axis is $\alpha = 7°\text{-}23'$; the cross-sectional area of the metal of the pipe is 71.2 sq. ins., and it was known from observations that the temperature of the water at the bottom of the lake ranged from 39° to 57° F., whereby the contractile stress would be 3,024 lbs. per square inch, and thus produce a longitudinal pull of 215,309 lbs.; hence the average pressure on the lead bearings, under this pull, becomes 2,560 lbs. per square inch. The work of laying the pipe was actually done at a time when the water at the bottom was considerably cooler than 57° F., and therefore the bearing stress is correspondingly less.

It may be added that to avoid danger of loosening during transportation, the lead was locked into the channel rings by flowing while melted into 48 equidistant holes, ¼ in. in diameter, bored through each flange; and all the rings were carefully examined for soundness of the lead. The diametral steel flanges were connected with 18 bolts ⅞ in. in diameter, each of which was regarded as having a sectional area of 0.5 sq. in. at base of threads, and hence easily able collectively to resist the longitudinal pull computed above, and the same remark applies also to the shear on the 48 rivets through the bearing rings.

The foregoing has been written with the view of pointing out that flexible joints of large water pipes are capable of rational computation with regard to the requisite strength of the component parts. So far as the writer has been able to learn, no other designs than those submitted above have been published, and many of them are defective in one or more respects. To make such joints permanently watertight is a difficult matter, especially if motion takes place frequently after the work is completed, and the best result will probably be attained by designs in which the bearing surfaces are of hard non-corrosive and highly elastic metal.

It is clear that a badly-rusted surface will cause serious damage to any soft packing material if any attempt is made in after years to change greatly the existing deflections in a line of submerged pipe. Lead should not be subjected to an excessive compressive stress, as otherwise it is liable to flow, in which event leakage will ensue after relaxation of longitudinal pull by rise of temperature, etc., sets in. Packings of organic material are generally objectionable, as they sometimes decay when kept in water for a long time, besides losing their resilience when subjected to great pressure. Accuracy of workmanship and smoothness of spherical surface is also necessary to avoid abrasion and consequent leakage. Vertical settlements of the line of pipe should be prevented as far as possible by providing a firm foundation or support, and in doubtful ground it is expedient to provide ample strength of all the parts, as the longitudinal pull may then become very great; the limit in this case being obviously the tensile strength of the pipe itself.

Little has been said in the foregoing about the work of laying submerged pipes, as this requires far more space than is now available. Many different methods have been used, and a good description of them cannot fail to be of great interest to those who have such work in charge. Much might also be said about the employment of divers to make submerged joints, and the precautions that are necessary to insure good results. In general this kind of work is extremely difficult, and the writer has therefore placed more dependence on well-designed joints that are put together under proper supervision by competent mechanics above water, than on those made with much trouble by divers whose workmanship can be proved only by careful and long-continued tests.

The method of computing the stresses in the components of a spherical joint has already been indicated. The probable longitudinal pull due to construction of the pipe by fall of temperature, etc., should first be determined, and after this the permissible maximum compressive stress on the packing material is to be chosen. The latter in turn will govern the area and overlap of the bearings, as well as the thicknesses of the ball, clamp-ring and flange of the bell, and its action on these members must be compared with the stress resulting from the leverage of the spigot or ball in the socket or bell while the pipes are being laid. This latter stress may become very large and hence an effort is made to keep it within a reasonable limit by reducing the number of pipes to be supported by a single joint. The conditions are shown diagrammatically in Fig. 30, where A, B and C denote three pipes and bells, of which A and C are supported, while the joint B is free. Let W denote the weight of one length L of pipe, then from the common theory of flexure, equilibrium is maintained when the moment WL is balanced by the two equal forces S acting within the bell B on the lever-arm b; hence in this particular case

$$Sb = WL \quad \text{and} \quad S = \frac{WL}{b}$$

We thus have the magnitude of the force S, which is the resultant of all the component vertical forces acting on one-half of the circular bearing whose mean diameter is d. These component forces, however, vary in

Fig. 31. (a) Component Vertical Forces Acting on One-Half of Circular Bearing of Mean Diameter d. (b) Semi-Ellipse Area of Which Equals Resultant of Foregoing Components.

magnitude from E to F, as indicated in Fig. 31a, and it may be assumed that the resultant S can be fairly represented by the area of the semi-ellipse $E\,G\,H$, Fig. 31b, whose semi-axes are $E\,F = \frac{d}{2}$ and $F\,G = c\,q$, where c denotes the width of the bearing and q the maximum pressure per unit of area. On this basis we will have

$$S = \left(\frac{\pi cqd}{4}\right) \text{ and } q = \left(\frac{4S}{\pi cd}\right) = \left(\frac{4WL}{\pi bcd}\right).$$

For example, in the case of 60-in. Rochester intake mentioned above, the figures were $W = 28{,}000$ lbs., $L = 1{,}320$ ins., $d = 69.42$ ins., $b = 9$ ins., and $c = 3$ ins ; hence

$$q = \left(\frac{4}{\pi} \times \frac{28{,}000 \cdot 1{,}320}{9 \cdot 3 \times 69.42}\right)$$

$= 25{,}107$ lbs. per square inch on the bearing. This was obviously inadmissible, and therefore provision was made for supporting the joints B and C, so that the weight on the two semi-circular bearings of a joint would be only about 14,000 lbs. and

$$q = \left(\frac{4}{\pi} \cdot \frac{14{,}000}{2 \cdot 3 \quad 69.42}\right)$$

$= 42.8$ lbs. per square inch, or twice this intensity if the ball rested on only one of the two lead rings.

To determine the stress on the bearings when a longitudinal pull P acts, use is made of the parallelogram of forces shown in Fig. 32. From the overlap $DA = n$, and the radius $CA = r$, the angle $DCA = \alpha$ is found from $r \sin \alpha = n$ and $\sin \alpha = \frac{n}{r}$; the mean diameter $AB = d_1$ of the bearing surface is derived from $d_1 = 2\,r \cos \alpha$; hence if c denotes the width of the bearing, the area of the latter will be $A = \pi d_1 c$, and the proportional part of the pull P that acts on each unit of area of the bearing will be: $p = \frac{P}{A}$. For the sake of simplicity, it will be assumed that this surface is a relatively narrow conical zone, instead of a spherical one, so that all the elementary longitudinal forces p will be equal, and can each be considered as the resultant of a transverse force t and a normal force q, the magnitude of the latter being

$$q = \left(\frac{p}{\sin \alpha}\right) = \left(\frac{P}{2\pi cr \cdot \sin \alpha \times \cos \alpha}\right) = \left(\frac{P}{2\pi cn \cdot \cos \alpha}\right) = \frac{P}{2\,\pi cn\sqrt{1 - \frac{n^2}{r^2}}}$$

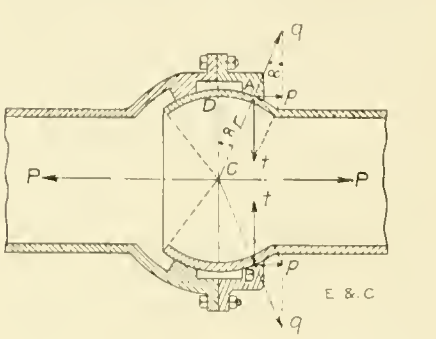


Fig. 32. Notation and Force Diagram for Stress Due to Longitudinal Pull = P.

hence if q is given, we will have the relation

$$cn\sqrt{1 - \frac{n^2}{r^2}} = \left(\frac{P}{2\pi q}\right)$$

in which c, n and r are variables. It may be added that r depends on the values adopted for n and c, the required deflection, and the radius r_0 and thickness x of the pipe.

If there are no intermediate yielding joints between the flexible ones, the pull P will be determined by the contractile force due to the given range of temperature, y_0F; and if the cross-sectional area of the mild steel pipe is denoted by A_1 square inches, we will have: $P = 0.0000056 \times 30000000 \times y \times A_1 = 168yA_1$ and

$$q = \frac{168yA_1}{2\pi cn \cos \alpha} = \frac{26.74yA_1}{cn\sqrt{1 - \frac{n^2}{r^2}}}.$$

For cast iron the figures in the numerator will be different, depending on the values taken for the modulus of elasticity E and the coefficient of contraction. The latter may be considered to be the same as for mild steel, but the former is uncertain, as it appears to vary with the tensile stress applied; hence the writer can only state that some good authorities give a value of $E = 15{,}000{,}000$ lbs. per square inch for this material in tension and compression.

The expansive stress produced in the bell and clamp-ring by the pull P is governed by the action of the uniform transverse forces t, which are the vertical components of the aforesaid forces q, the relation being expressed by $t = q \times \cos \alpha = \frac{p}{\tan \alpha}$, per unit of area of the bearing surface; and if these rings are wider or narrower than the horizontal width $c \cos \alpha$ of the bearing, the uniform expansive pressures must be correspondingly reduced or increased to t_1 per unit of area. For the sake of simplicity, it will here be assumed that the rings have a width c_1, an average depth or thickness z_1, an internal mean diameter d_2, and a permitted tensile stress of s lbs. per square inch of the sectional area c_1z_1; hence the necessary thickness z_1 of the ring will be found approximately from the familiar formula for thin pipes: $z_1 = \frac{t_1 d_2}{2\,s}$. If the thickness is relatively large, a modification of this expression is necessary, whereby the value of z_1 will be somewhat increased.

It should be remembered that the foregoing formulas have been based on a conical bearing-surface in order to facilitate their development. Strictly considered, the actual spherical surface should be taken into account, but as this leads to complex mathematical expressions, the writer has omitted this part of the subject. Attention should also be called to the fact that the various figures shown herewith are not drawn to scale.

Bridges

Some Design and Erection Features of the Superstructure of the Salmon Bay Bascule Bridge in Seattle, Wash.

Contributed by H. A. Gerst, Bridge Dept., G. N. Ry., St. Paul, Minn.

The Salmon Bay bascule bridge is located on the Great Northern Ry.'s new main line between Metum and Interbay (Seattle), Wash. The bridge is about six miles from the Great Northern Ry.'s King St. passenger station in Seattle. Its axis lies almost due north and south and crosses the Salmon Bay waterway at an angle of 67°. The government requires a clear width of opening of 150 ft., perpendicular to the axis of the channel, at an elevation of 25 ft. below extreme low tide. To obtain this clearance it was necessary to make the length of the movable leaf of the bascule span 200 ft. 0 ins. from the center of the main trunnions to the center of the skewed end floorbeam at the free end of the span. The upstream movable truss is 207 ft. long, and the downstream fixed truss is 193 ft. long. The movable leaf is the largest and heaviest of this type on the Pacific coast, and the bascule bridge, taken as a whole, is the second largest of its type in the country.

DESIGN FEATURES

In the spring of 1912 the railway company officials decided on the Strauss low trunnion type of bascule for this particular crossing. The structure is designed to carry two tracks spaced 11 ft. 0 ins. on centers. The trusses are spaced 32 ft. 0 ins. on centers and afford a 7-ft. 6-in. side clearance from the center line of the track to the inner edge of the trusses. The American Railway Engineering Association's general specifications were used for the design of the structural parts of the bridge, the live load assumed being Cooper's E-55 loading. Slight modifications were adopted in the matter of dead load stresses, however, and a dead load tensile stress of 22,000 lbs. per square inch was allowed in combination with live load tensile stresses. When dead load compressive stresses were considered the dead load stress was decreased three-elevenths, and the allowable stress was taken at its usual value. The specifications of the Great Northern Ry. for steel structures were used for materials and for their fabrication, except as modified and supplemented by a special set of specifications furnished by the Strauss company.

The bridge, when open, was designed to withstand a wind pressure of 15 lbs. per square foot acting in any direction, and the operating machinery was made strong enough to hold the span stationary against this wind pressure. Two main electric motors each of 100 HP capacity were designed to open or close the movable leaf through an arc of 83° in 1¼ minutes, against a wind pressure of 2½ lbs. per square foot. These motors were also of sufficient capacity to operate the movable leaf against a wind pressure of 10 lbs. per square foot at a slower speed. The maximum torque due to the 15-lb. wind pressure amounted to 6,955,500 ft. lbs., which caused a tooth pressure on each of the two operating racks of 116,000 lbs. The 10-lb. wind pressure caused a torque of 2,100 ft. lbs. at each of the main operating motors. Alternating current, 3-phase, 60-cycle and 110-volt, is used for the electric motors. A 5-HP electric motor is used for operating the end latches, with a hand-power auxiliary provided for emergencies. A 25-HP. gasoline engine and also a hand-operated gear are provided for use in raising and lowering the bascule leaf if for any reason the electric power is temporarily cut off. The unit stresses used in the calculation of the size of the machinery parts were as follows:

Item –	Stress, lbs. per sq. in.
Slow speed shafting	16,000
High speed shafting	12,500
Slow speed gears	17,500
Medium speed gears	12,500
High speed gears	9,000

The trunnion bearings were designed for a maximum pressure of 2,000 lbs. per square inch. The counterweight was designed to rotate about a point 53 ft. 0 ins. above the top of the ties, the counterweight trunnion being 31 ins. in diameter. The extreme length of the counterweight frame is about 100 ft. The total movable weight of the bascule is about 1,700,000 lbs.

Emergency air brakes, with cast-iron shoes actuated by a toggle mechanism, are provided for on each of the two operating struts. The air brakes, which are attached to the strut guides, swivel on the outer ends of the main operating or rack pinion shafts. Compressed air is supplied to the cylinders of the air brakes at a pressure of 100 lbs. per square inch by an electric, direct current air compressor of 3-H.P. nominal rating. An air buffer is also provided on the free end of the movable leaf. This consists of a cylindrical plunger which comes in contact with a 1-in. thick steel plate on the concrete rest pier.

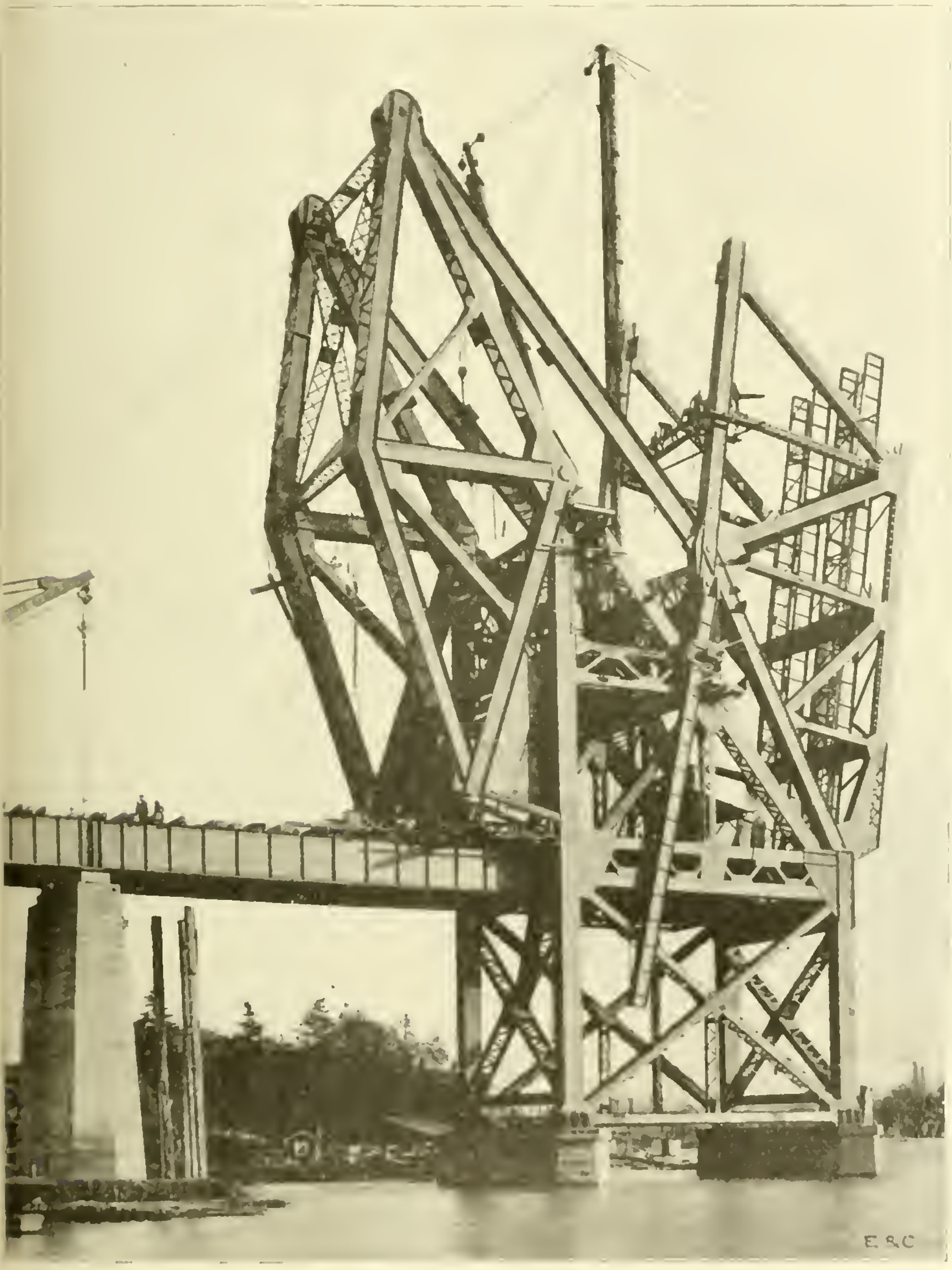


Fig. 1. Progress View of Salmon Bay Bascule Bridge, Seattle, Wash., Showing Method Used for Erection.

The operating machinery and all motors, except the end latch motor, are located on the stationary part of the bascule, being placed in a sheet steel house having a reinforced concrete floor. The bridge is operated from a two-story house, 10x12 ft. in plan, where the necessary controllers, switchboard and mechanical interlocking stand are placed.

ERECTION FEATURES

The tower, including the two 25-ft. tower spans, was erected by means of a derrick car of 25 tons capacity having a 50-ft. boom. The counterweight girders, the side plates and the lower portion of counterweight trusses were raised into position by means of the derrick car, assisted by a guy derrick, which was placed on the machinery floor about 27 ft. above the deck. This guy-derrick had a 100-ft. mast and a 90-ft. boom, and could handle, alone, the heaviest piece, which weighed about 25 tons. It was not called upon to handle, singly, more than 22 tons, the weight of one counterweight link. The first member set in the counterweight truss was so placed (its position being fixed by calculation) that the movable leaf portion, which was erected in front on the main trunnion, was not disturbed during the insertion of the two counterweight links. The movable leaf was erected in a nearly vertical position, its axis being only 7° 30′ from the vertical.

After the counterweight trusses and about 70 ft. of the movable leaf were erected the placing of the 735 cu. yds. of concrete in the counterweight was begun. The lagging for the concrete block was placed on a 12x12-in. cribbing, laid crosswise. This lagging, which consisted of 2x10-in. fir planks, was held tightly in place by means of pairs of wedges. As the concrete block was only a few feet from the deck, when the span was in the open position a substantial support was obtained by placing the cribbing directly on the floor stringers and girders. The side plates, which were ⅜ in. in thickness and had rim angles turned inward, conformed to the outline of the concrete. Wooden forms were required only on the top, bottom and ends. The concrete used in the counterweight was a 1:2½:5 mix of Portland cement, sand, and gravel, test blocks being made beforehand to insure that the weight of 148 lbs. per cubic foot was approximately attained. The center-of-gravity calculations were made on the basis of this weight. The sand and gravel were obtained from Richmond beach, which is several miles from the bridge site. The grading of aggregates ran very uniform in sizes. The bottom 5 ft. of the concrete counterweight were poured practically continuously. This portion was then allowed to set for 36 hours, which tended to develop slab action and to relieve the supporting floor system underneath the lagging of a considerable part of the load caused by the pouring of the remainder of the concrete. Figure 1 shows a view of the partially erected bascule span and the counterweight girders. The view was taken after the bottom 5 ft. of the concrete counterweight had been poured and allowed to set, and shows the guy-derrick in place.

Eight adjustment pockets were left in the counterweight monolith, these pockets providing space for 832 adjustment blocks should the same ever be required. Each adjustment block weighs about 160 lbs., and under normal condition it is estimated that about one-half of the total number of blocks will be required in the counterweight. As the concrete counterweight is about 21x34x28 ft. in size it will require several months before it is cured sufficiently to enable the final adjustments to be made.

A hoisting tower, measuring 127 ft. from base to top, was erected, and a suspension cable was strung from the bridge over the top of the tower and down to a "dead-man" to the opposite side. From this cable there was suspended a rectangular wooden chute for spouting the concrete into place. It was found that the most desirable slope for this

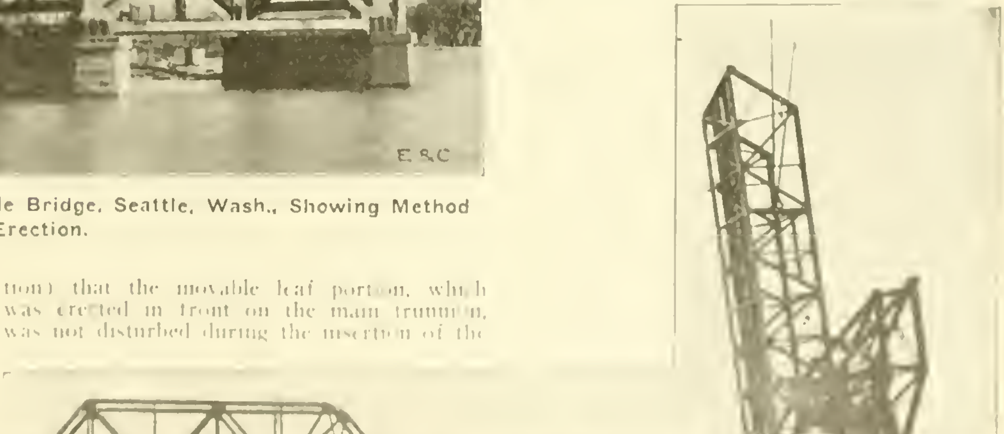


Fig. 2. Lift Span Practically Completed In Raised Position—View Also Shows Fixed Span.

chute was 1 to 3½, as the coarse aggregate did not "run away" on this slope. The bottom and sides of the chute were built of surfaced fir planks. The skip was hoisted by means of a 35-HP. electric motor, and the concrete mixer was operated by a 10-HP. motor. Barring delays, about 40 ½-cu. yd. batches of concrete per hour could be mixed and placed by the concreting plant.

The movable leaf was erected by two additional "jumps" of the guy-derrick. Figure 2 shows a view of the lift span practically completed, with the guy-derrick in its highest position. The view also shows the adjacent fixed span. The principal advantage of the guy-derrick scheme is that the various parts of the structure can be picked up from behind the counterweight and trusses by "booming out," and can then be placed in the structure by turning the derrick through the required angle. In the A-frame or stiff-leg derrick scheme, their moving up does not require as much time, but a material track would have been necessary alongside of the movable leaf if this method of erection had been adopted. As the distance from low tide to top of tie is 65 ft. a trestle would have been very expensive.

The bascule structure contains about 1,155 tons of steel, not including motors and appurtenances, and required approximately 40,000 field rivets and bolts. The assembling, erecting and riveting of this bridge required an average force of 22 bridgemen for about six months. Fir ties, surfaced on all four sides and dipped in "Carbolineum" paint, were used for the floor system of the bascule, tower, and adjacent spans, except the north and south approaches, which are of ballasted deck construction. Figure 3 shows a view of the completed bridge in the closed position. The view also shows some of the pockets left in the concrete counterweight for adjustment purposes.

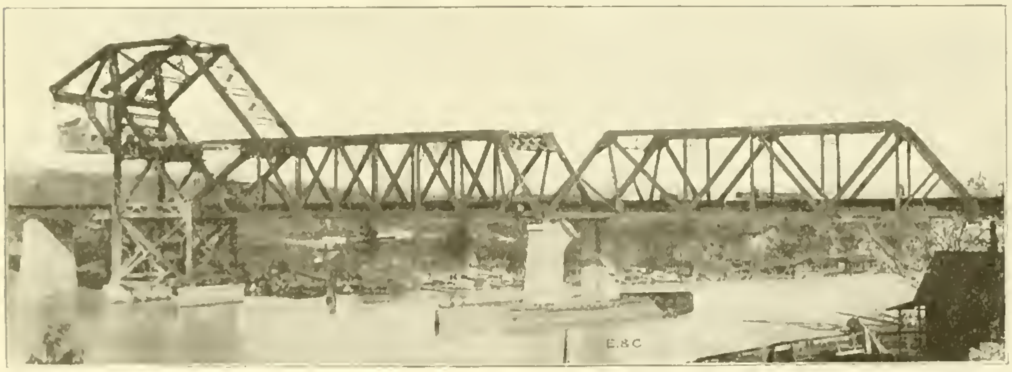


Fig. 3. View of Completed Salmon Bay Bascule Bridge—Movable Span in Closed Position.

PERSONNEL.

The structural steelwork and the machinery were furnished by the Fort Pitt Bridge Works of Pittsburgh, and were erected by the Gerrick & Gerrick Co. of Seattle. The Strauss Bascule Bridge Co. of Chicago designed the bridge. The bascule and its approaches, including a 165-ft. double-track span, were constructed under the direction of Mr. Ralph Budd, chief engineer, and Mr. John A. Bohland, bridge engineer, Great Northern Ry. The writer was in direct charge of the erection of the south approach, the 165-ft. span and the bascule span.

Hydraulics Laboratory, University of Illinois.—The Hydraulics Laboratory of the University has just added another piece of apparatus to its equipment. There is being installed a 1-in. 3-stage De Laval Centrifugal Pump. The pump will be driven by a 50-HP. Westinghouse variable speed induction motor and will be capable of giving a discharge of 275 gals. per minute against a head of 100 ft. The pump will be arranged so that water can be drawn from the laboratory standpipe into the pump and in this manner the total available head of 150 ft. can be had.

Painting of and Paints for Steel Railroad Bridges.

To obtain satisfactory results in painting steel railroad bridges requires not only the use of good paints but a proper application of the paints. Different parts of the bridge are subjected to different exposures, and this fact must be taken into consideration. The comparatively short life of railroad bridges under existing conditions is sufficient evidence that the subject of paints and painting is an extremely important one. The following data on the difficulties and deficiencies of railway bridge painting were taken from a paper by Mr. W. G. Wilson, presented before the recent convention of the Railway Bridge and Building Painters.

Even though paints and paint materials are now made better than ever before, conditions are changing, and every year adds to the difficulties of those in charge of painting steel railroad bridges. The object of this paper is to deal with some of the causes of paint deterioration and the consequent rusting of the steel. It should be mentioned that conditions as bad as we now have to contend with have probably come to stay, and we must make every effort to meet them. The railway foreman of painters can, by careful observation, analyze the conditions, and by common sense methods do much to improve the situation.

Different surfaces, different climates, different exposures and different conditions of application require different paints and compositions. The greatest mistake in railway bridge painting is the use of any one particular kind of paint to meet all conditions on all parts of a structure alike or on all bridges regardless of locations or conditions.

Good linseed oil and pigment will, when applied to all parts of a bridge alike, give splendid protection to such parts as have sunlight and open air exposure, and may utterly fail on such parts as have wet surfaces, or are exposed to salt brine, smoke or acids.

A bridge built at a high elevation in open dry air and sunlight and of open truss construction can be easily protected with any good linseed oil paint. A bridge built low over water, swamp or marshy ground, of riveted plate girder construction, carrying heavy freight traffic on main lines cannot be protected on all parts with a pure linseed oil paint. This kind of a bridge requires special attention, with a special moisture proof paint. If this bridge is painted with linseed oil paint the upright parts may be in good condition two, three or four years after painting. The horizontal parts, tops of chords, tie plates, floorbeams, struts and gusset plates, under severe conditions, will rust within a few months, and some one condemns the paint and tries another and another of the same nature, but probably another firm's make, only to meet the same results. Now the fault is not the paint at all, but that of the engineer or painter. The painter must solve the difficulty, but to do so he will have to resort to something different than the ordinary way. Find out why paint has failed—and right here is where the test of efficiency comes in. The railway maintenance of way painter can find the way and save the destruction of the bridge if he is a bridge painter as well as a house painter, and he must be both to be equal to the job.

What are the causes of paint failing to protect only on parts of the structure? Difference in exposure, nothing else. Moisture and oxygen destroys the iron or steel and the problem is to keep the moisture away from the iron and steel. It may be difficult, but you can do it. Paint to protect, not merely to coat over, as many contractors do. Work as the railroad painter, not as the contractor.

Painting may be preservation of structure or destruction of structure. I have seen plenty of it done both ways. Every square foot of surface which the contractor can avoid cleaning and removing rust from is money in his pocket. It does not matter much to him if the surface is wet or dry. If the inspector is not looking his men will paint it over.

In a good many instances the inspector is a railroad carpenter, not a painter, and any place that is covered with paint is painted, so far as he is concerned. He can understand the quantity of paint put on, but does not understand properly rubbing and working it out. In cases of this kind the work is sure to be defective.

Another deficiency originates from the practice of allowing carpenters to paint the tops of bridges when laying new bridge ties. The painter will have a standing order to supply the paint to the division general foreman of carpenters, who has orders to paint the tops of all bridges when laying ties. This work is done at all seasons of the year and in all kinds of weather. The carpenters have no proper tools for cleaning the rust from the iron. Probably a wire brush and broom. If the rust is very thick some of the coarser scales may be scraped off. Snow and ice may be only partially removed, or there often is a drizzling rain falling. No matter, arrangements have been made to run trains over the other track. The ties must be laid and the top of the bridge must have a heavy slushing of paint regardless of conditions. The paint cannot dry or fasten itself to the iron under such treatment and serves only to catch dirt, sand, ashes and cinders; this when a scattering of brine from refrigerator cars is sprinkled into it, makes a salt pad to hold moisture and promote rust. The intent of the order is good, but the outcome could not be worse. The remedy: Leave the application of paint to the painter and allow no paint to be applied to iron or steel, except under the most favorable conditions.

Every condition brings up against the greatest difficulty of all—corrosion, rust. We paint to preserve. Then let us be very careful or we may paint to destroy—and defeat the very purpose of painting. Protection of railway iron or steel in bridges is a very complicated matter. Corrosion, we can easily account for, as it is simple and easy to understand, involving the presence together of free oxygen and water. The chemical, or, more strictly speaking, electro-chemical reactions may be divided into two stages: First, solution of a small portion of iron, and, secondly, oxidation of this iron to ferric hydroxide or iron rust. The red oxide, thus formed, is insoluble; therefore, when precipitated, it leaves the way clear for more iron to enter the solution and for the further combination of iron with oxygen. These conditions will continue so long as there is any oxygen, water or iron left. The absence of either water or free oxygen makes corrosion impossible.

The rate of corrosion depends among other things on the amount of each of the elements present. It depends principally on temperature and the character of the water, galvanic action between different materials, electrolysis from stray currents and many other minor influences. It would be simple enough if we had only oxygen, water, galvanic action, etc. alone to deal with, for then any good linseed oil and pigment would preserve the structure. Right here is where the difficulties thicken and multiply. Things that dissolve and destroy paint are numerous, sulphurous acid gas, carbonic acid gas, salt brine, cinders, dirt, coal dust drippings from soft coal cars, etc.

Linseed oil, the most desirable vehicle for all paints to be used on wood, on account of being porous to some extent is very susceptible to the destructive influences of these destroyers of paint. After carefully investigating, so far as limited means would allow, it is my opinion that a prepared paint vehicle made of linseed oil reinforced with China wood oil is a very great improvement in waterproofing the paint. I have tried several kinds of paints said to contain China wood oil, and some paint oils said to contain China wood oil, and have found that they would dry quicker, with a hardness and non-porosity that would outwear linseed oil under hard exposure. Tests were made on bridges, painting parts with the same salt brine exposure or exposure to smoke, engine exhaust and gases. The result was that where linseed oil paint lasted well for three to four months, linseed and China wood oil paints lasted one year in good condition. On a bridge, where the floor system was painted two coats of red lead and two coats of black carbon, salt brine exposure had destroyed the linseed oil paint in six months, with the same exposure. China wood oil paint stood two years in fairly good condition, and parts of the same floor system, painted with a patent moisture-proof paint stood two years in good condition. A mixture composed of 1 gal. of heavy red lead in linseed oil, 1 pint of heavy No. 96 liquid gum binder, ¾ gal. of asphaltum varnish, ¼ gal. of extra car varnish was applied to the tops of tie plates, floorbeams, struts and brace plates, after thorough cleaning; it dried in splendid condition and allowed the application of two heavy coats in one day under exposure that would destroy straight linseed oil paint before it could dry at all. On parts of this same bridge, not subjected to direct moisture and brine exposure, black carbon paint lasted well, three to four years.

With a special reinforced linseed oil paint absorption of moisture can be overcome to such an extent that, by proper cleaning and touching up each year, all parts of the structure will be preserved alike. On main line tracks, under bad exposure, all bridges should and must be carefully cleaned and touched up by a competent man, with a heavy moisture-proof reinforced oil paint, at least once a year. A light expense which will save one-half to three-fourths of the present cost of railway bridge painting. I am writing from actual experience. My plan is, when a bridge is in good condition, keep it so. It is the only way to paint to preserve the structure.

Ordinarily 75 per cent of the cost of bridge painting is for cleaning; 60 per cent of the cleaning can be eliminated. That means cutting the regular cost of bridge painting in half.

It is commonly granted that red lead and linseed oil make the best priming coat and filler for bridge painting; but to all rules there are exceptions.

The writer knows of large bridges subjected to excessive salt brine drippings, coal dirt, cinders, soft coal and ashes, into which hot and cold water from shifting engines is constantly being dripped. To make a bad matter worse, engines from trains on tracks passing under the bridge subject it to the action of sulphuretted hydrogen and sulphuric acid. These agencies quickly destroy red lead and oil, which is about the worst material that can be used in parts of structures subjected to these conditions or in train sheds. The rusting of surfaces subjected to these conditions is greatly accelerated by galvanic action, which causes pitting and destroys the lighter members of the structure, thus necessitating constant replacing and much expense for repairs. There are cheap non-drying black japan elastic roof paints which will protect under these conditions, and by careful cleaning and painting once or twice a year will preserve the structure. If a cheap non-drying black japan roof paint will give better results, in these situations, why cannot a paint made of better materials and with the ultimate object of protecting iron under severe conditions help solve the difficulty?

There are several specialties made for this very purpose by chemists and paint makers who are eminent authorities. Special non-porous, non-absorbent paints that lock up the pores of any surface to which they are applied; paints that do not dry by oxidation, and contain no tar. Special paints for special purposes will greatly aid in the preservation of structures under severe conditions.

Substructure Work for the Fall River Bridge and Repairs Required by a Partial Failure of its Foundation.

The following article treats of the methods used in constructing a highway bridge in deep water, of the partial failure of its foundation, and of the methods used to strengthen it. The data were taken from a paper by James W. Rollins, presented before the Boston Society of Civil Engineers. The original contract was let in December, 1906, and the bridge was opened to traffic in the spring of 1908. In July, 1911, some of the piers settled and tipped badly, making it necessary to reconstruct their foundations. After an investigation it was decided to repair the foundations of some of the piers and to reconstruct those of others, and this work was accordingly done under a separate contract by the original contractors.

DESCRIPTION OF BRIDGE AND ORIGINAL SUBSTRUCTURE WORK.

The Fall River bridge, which spans the Taunton Great River at Fall River, Mass., is a highway structure with a 50-ft. roadway and two 10-ft. sidewalks. It is a deck bridge and consists of three 200-ft. spans, two 100-ft. spans, and a double-leaf Scherzer rolling lift span with a 100 ft. opening.

The contract drawing for the piers was merely a cross-section of the river, showing the position of its bottom and the location and size of each pier, with no further details except an indication as to whether the footings were to rest on rock or on pile foundations (see Fig. 1).

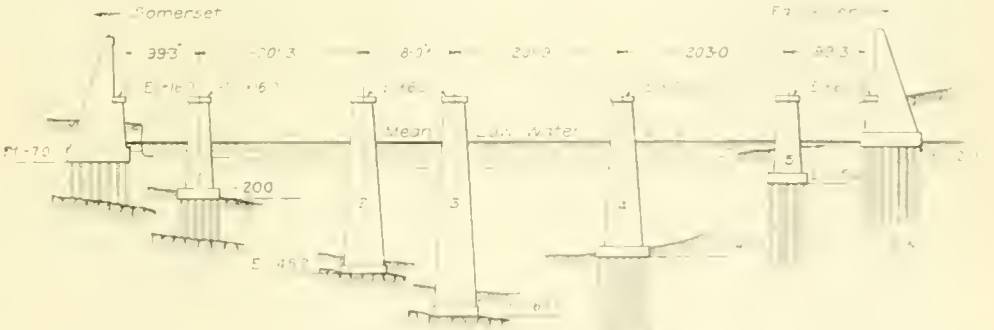


Fig. 1. Contract Drawing for Piers of Fall River Bridge, Showing Paucity of Data Furnished to Contractors.

The specifications provided that "Piers 1, 2 and 3 shall be placed on foundations prepared on firm, solid ledge. Pier 4 shall be built on a foundation of piles cut off 10 ft. below low water, Pier 5 on piles cut off 15 ft. below low water."

It was evident from an examination of the river section and from a consideration of the requirements of the specifications that piers Nos. 2 and 3 must be built on pneumatic foundations, as rock was about 60 ft. below low water. After consulting with the engineer the contractors decided to build piers Nos. 1, 4 and 5 on pile foundations, and piers Nos. 2 and 3 on pneumatic foundations. Working plans were accordingly made for constructing the piers which were to have pile foundations by means of open caissons and heavy wooden grillages.

For each caisson a timber grillage was built of 12x12-in. yellow pine timbers extending about 1 ft. in each direction beyond the masonry foundation (see Fig. 2). The first course of this grillage was longitudinal, the second made an angle of 45° with the first course, the third was at right angles to the second course, and the fourth was at right angles to the first course. The timbers were fastened together with 1x20-in. drift bolts at each intersection. These timbers were sized one way to make the courses of even thickness. Hook bolts, made of 1¼-in. iron rods, were run through the two top courses and fastened every 4 ft. They were so shaped at the top that the loops in the ends of the vertical rods holding the sides of the caisson to the bottom would drop clear of the hooks when the rods were unfastened and driven down after the masonry was completed. The top course of the grillage was then thoroughly calked in both horizontal and end joints with oakum and pitch, and the grillage was launched, it having been built on timber launching ways and floated to the derrick or lighter which was to be used for constructing the sides.

The sides of the caisson were built of 6x12-in. vertical posts spaced 24 ins. on centers, and planked with 3-in. or 4-in. planking, depending on the depth of the water (see Fig. 2). These posts were from 16 to 24 ft. high, varying with the depth of the water and the size of the caisson, it being considered undesirable to have more than one joint in the sides. The height of the sides was limited by the weight—from 20 to 30 tons—which could readily be handled by the lighter or derrick.

On top of the 6x12-in. vertical posts were spiked 6x12-in. caps, and 1¼-in. rods, with turnbuckles and with nuts at the top ends and loops at the lower ends, were placed with the loops over the hook bolts in the grillage, these holding the sides down firmly to the grillage. All the joints in the sides and ends were thoroughly calked. The sides and ends were then thoroughly braced and cross braced to keep the caisson in shape and to withstand the water pressure until the masonry was built.

The masonry was then laid, beginning at the center and working towards the ends. When the weight of the masonry sunk the box in the water to within 1 ft. of its top the second set of sides, made similar to the bottom set, were hoisted into place, the posts of the second set being fastened to the cap on the first and then calked tightly and braced as before. This process was continued with the third and fourth sets of sides, until the bottom of the loaded caisson at high tide floated about 1 or 2 ft. above the tops of the piles upon which it was to rest. Generally pile dolphins are driven to keep the caissons in position, but at Fall River the water was too deep. Consequently heavy anchors, consisting of 15-ton concrete mooring blocks, were used.

After the caisson was lined up both ways by the engineers and leveled it was settled at low water by knocking out the plugs which had previously been put in below the water line, and was allowed to sink upon the pile foundation. Additional masonry was then laid as rapidly as possible to add to the weight of the caisson, but it was not placed above the low water mark. If the masonry inside the caisson was below the outside water level when the caisson was sunk, the plugs were put back, were calked, and then the water inside the caisson was pumped down below the masonry level. At this time the lines and levels were again run, and the masonry was trued up to its proper position, the bottom section which was built when the caisson was afloat having been made large enough to afford an offset of about 6 ins. at the top, thus giving sufficient room for a true adjustment of the pier above the water line. The masonry was then finished above high water.

As the masonry was built up the braces were taken out and the sides of the caisson were maintained by short braces placed from them to the stonework. When the last side was in position long rods, placed from the top cap to the turnbuckle below the cap in the first set, were substituted (one by one) for the short rod ends which extended from the turnbuckle to the first cap, so that the entire flotation pressure of the sides came upon these rods (see Fig. 2). To release the sides the top nuts were taken off, and the rods were driven down so that the loops in the lower ends were driven clear of the hook bolts in the grillage, thus freeing the sides, which floated up when released and were used again on other piers.

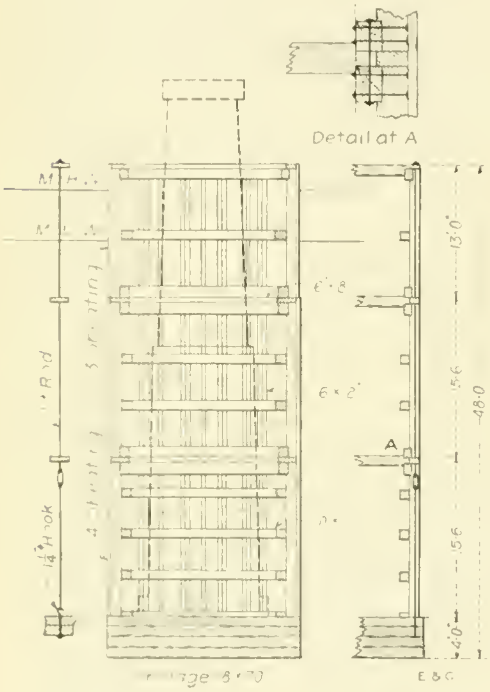


Fig. 2. Cross Sections of Caisson for Pier No. 4, Showing Type of Construction and Arrangement of Rods and Hooks—Fall River Bridge.

Both on account of the depth of water at the draw (about 60 ft.) and the small amount of earth covering above the ledge, riprap was ordered deposited around the fender piles to hold them in place and to help support them.

The contract for the bridge complete was awarded in December, 1906, and work was begun at once. All masonry work was completed by Oct. 1, 1907, and the steelwork and approaches were finished and the bridge opened for traffic in the spring of 1908.

FAILURE OF FOUNDATIONS AND REPAIR WORK REQUIRED.

On July 20, 1911, the writer was told that one of the piers of the Fall River Bridge had settled. He was asked to go to Fall River the next day to meet the county commissioners, to find out what had happened, and to determine what should be done. At this meeting it developed that pier No. 4 had settled 2 ft. at the bridge seat level at the south end, about 8½ ins. at the north end, had moved upstream to the north about 12 ins., and had tipped 5 ins. to the east.

Various theories were advanced as to the cause of the failure. At that time the writer held that the piles had been crippled in driving and had been broken off, as the bottom was practically hardpan. Having been in actual charge for the contractors, and having full knowledge of all details of construction, he offered to investigate and report to the county commissioners as to the cause of trouble. This offer was accepted and he was given instuctions "to do all necessary work to protect temporarily and to preserve said bridge."

Divers went down and reported that the piles were broken off, and that the pier was resting on them. Another diver of great experience was sent down and he reported in part that the filling had been scoured out around the piles to a depth of 2 to 3 ft., thus exposing the piles, many of which at the south end of the pier had been broken off at the ground line. He brought up a piece of one of these piles and reported that "this portion of the pile bears evidence of the ravages of the teredo and has been eaten away to such an extent that but little of the pile was left at the time of the breakage, which explains in a large measure the cause of the pier's failure." This report was made on July 27, and was the first intimation of the true cause of the failure—namely, the ravages of the teredo. It was common knowledge that the teredo was to be found in Taunton Great River and was very active on the wooden structures in that vicinity, but no one thought of the possibility of its working 15 ft. below the water line.

The first conclusion was that the pier was resting on the crippled piles and on solid bottom, careful observation having been made daily and no further settlement having been noted. Plans were then made to jack up the trusses on each side of the pier and to block them in their true line and grade, so as to permit the bridge to be opened to traffic. In studying this plan it was found that the trusses weighed 500,000 lbs. each for one span and about 400,000 lbs. for the other span, and that there was no room on the pier to place jacks and nothing in the steel construction at the pier strong enough to hold the dead load of the bridge on the jacks.

Further study of conditions showed a great uncertainty of stresses and resistance to them. Pier No. 3 was tipped 4 ins. to the east—*i. e.*, along the line of the bridge; spans Nos. 4 and 5 were "butted" against each other on pier No. 5, and span No. 5 was solidly against the Fall River abutment; so that the stability of pier No. 4 would be uncertain if the load of span No. 4 was taken off. Consequently, to avoid the danger of losing span No. 4 should pier No. 4 collapse, and also to provide means to jack up that span into its original position after repairing the pier, it was decided to drive piles under the panel point nearest to pier No. 4, about 40 ft. away, and to place blocking on these pile bents close enough to support span No. 4 should it settle further. The water at this point was 40 ft. deep, and the bottom of the river was hardpan. Therefore the heaviest piles obtainable—60-ft. yellow pine piles with 8-in. tips—were driven in groups of 24 under each truss, using a pile hammer hung on leads suspended from the top chords of the trusses. This proved very slow and difficult work.

On account of a long legal controversy between the city of Fall River and the county commissioners, the latter being in authority in the matter, several weeks elapsed before it was decided what should be done to protect the pier and who should do the work.

To prevent further settlement of pier No. 4 the writer advised the dumping of crushed stone around the foundation and the employment of a diver to push it back under the grillage as far as possible. A temporary foundation would thus be formed, as the bottom of the river was hard, with no trace of silt.

Acting upon the advice of the writer to get the opinions of experts, the county commissioners secured the services of an engineer—an expert on deep foundation work—and the city of Fall River retained two consulting engineers. The consulting engineers for the city and the writer, working independently, developed almost identical plans, which were adopted. These plans called for the following work:

The construction of a bulkhead around the pier, on the outside of which was to be placed crushed stone and riprap. Between the bulkhead and the grillage rich concrete was to be deposited, and then a rich cement grout under pressure was to be forced through pipes to fill all the space under the pier. For this work 6-in. wrought iron water pipes 6 ft. long were placed between the crippled piles under the grillage. Elbows were put on these pipes, and vertical pipes about 8 ft. long were fastened to them (see Fig. 3). A second set of grouting pipes 2 ins. in diameter was also put in to make sure that the space between the grillage and the river should be made solid.

The second operation consisted of building a form of 3-in. planks, about 8 ft. high, on a long timber frame made of 6x12-in. hard pine timbers, which extended 5 ft. above high water. The height of this form was about 50 ft. These frames were thoroughly cross braced, and were made in two sections on the sides, each about 30 ft. long, the ends being made in one piece.

The forms were weighted with rails, and were sunk 5 ft. outside of the grillages. A ¾x18-in. iron plate on the lower edge afforded a seal into the bottom of river or into the crushed stone which had been deposited around the pier. The forms were held rigidly by struts placed against the grillage, rods with turnbuckles being used to hold the forms up to the struts (see Fig. 3).

Riprap and crushed stone were then placed outside of the form up to its top, and 1:2:3 concrete was deposited through 8-in. tremies from the top of the bridge into the space between the grillage and the forms. This concrete was a wet mix and was brought up above the top of the grillage all around the pier. It was allowed to set, and then a rich concrete, composed of 1-in. pebbles, sand and cement, was forced into the 6-in. pipes; but with no great success, due probably to the fact that the concrete which was deposited outside of the grillage under considerable pressure had been forced into all the larger cavities under the grillage. The whole mass was then made solid by forcing grout through the 2-in. pipes until a pressure of 100 lbs. per square inch would produce no flow, after which a large quantity of riprap was deposited around the pier. The riprap was brought up to a greater height on the weak side of the pier (see Fig. 4).

LEGAL COMPLICATIONS CONCERNING RESPONSIBILITY OF FAILURE.

While all of this work was going on a very interesting legal controversy was being waged concerning the responsibility for the failure of this bridge, which had cost about $1,000,000 and had been built only two years. Its original cost was divided between the towns and cities interested, Fall River paying the largest share, about 55 per cent. The legislative act authorizing the bridge provided that the cost of maintenance and repairs should be divided between the city of Fall River and the towns of Somerset and Swansea and that 98 per cent of this cost should be paid by Fall River.

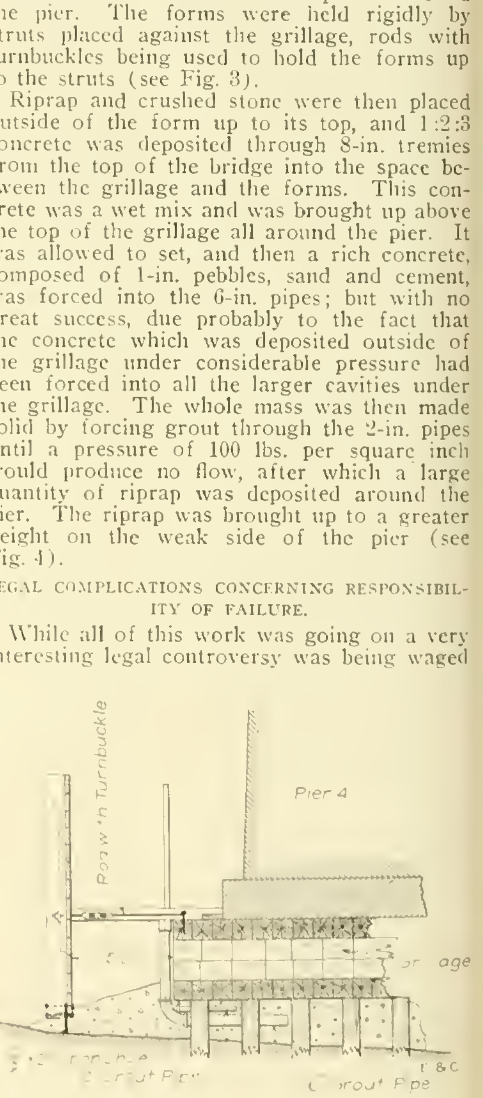


Fig. 3. Foundation for Pier No. 4, Showing Manner of Grouting Around Piles and Grillage—Fall River Bridge.

The plan of pier No. 4, the damaged pier, as furnished to the contractors by the engineer, called for piles to be sawed off at grade, elevation — 40.0 (see Fig. 1). They were then to be filled in to their tops with gravel or broken stone, and the pier was to be built in an open caisson resting on a 4-ft. wooden grillage consisting of 12x12-in. timbers. This construction was carried out, the pier was completed and the bridge was accepted and paid for.

On August 31 the engineer wrote to the joint board, advising the use of riprap at the fender, and stated that for protective purposes riprap should also be placed on the river side of the Somerset abutment and adjoining *piers Nos. 1 and 4.* On the same date he asked the contractors for a proposal to place riprap as above outlined, and a proposal was made to the joint board by them to place riprap for $3.50 per cubic yard, *to the extent required and as directed by the engineer.* This proposal was accepted on Sept. 1, and about 4,500 cu. yds. of riprap were placed about the fender pier, the Somerset abutment, and pier No. 1; but *none about piers Nos. 2, 3, 4 and 5.*

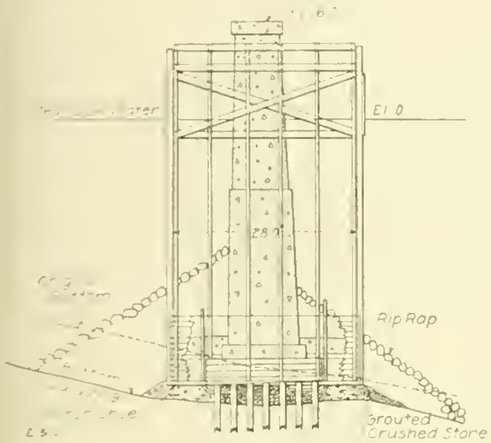


Fig. 4. Completed Repairs at Pier No. 4 of Fall River Bridge.

The writer had doubts about the sufficiency of the quantities of riprap placed, owing to the prevalence of the teredo, and wrote the resident engineer to that effect. However, the engineer did not think it necessary to riprap the piers. Consequently the work of riprapping was stopped, and the responsibility of not protecting the grillages and piles was placed on the engineer, who has never shirked this responsibility, or tried to place it, even in part, upon the contractors.

The actual cost of repair to all the piers, the work having been done by the contractors without any profit, was about $60,000. The repair work on pier No. 4 cost about $25,000, and was finally paid for by the state, and the bridge made a part of the state highway.

A question of great importance to engineers and contractors was brought up by this failure. The plan shown in Fig. 1 was all that was provided to show the construction of the piers, and the contract provided that "the contractor must inform the engineer as to the process he proposes to use in preparing the foundations and laying stone; also for driving piles, and must receive the approval of the engineer before beginning work." Under this provision the engineer presented plans to build piers Nos. 1, 4 and 5 on piles, in open caissons, and piers Nos. 2 and 3 on pneumatic foundations, and all work was done in accordance with these plans, under the direction and inspection of the engineers, and to their satisfaction, and was accepted by the commission. The contractors did not know, nor did they inquire as to the authority of the engineer to make these plans, but took it for granted he had the authority. The contract sketch plan, Fig. 1, indicates no particular method of construction, unless one infers an open cofferdam to rock in 60 ft. of water—a practical impossibility.

The contract further provided that "should it be found desirable by the engineer to make alterations in the form or character of any part of the work, he may, with the approval of the joint board, order such alterations to be made, defining them in writing and drawings, and they shall be made accordingly." Changes were made in pier No. 1, and it was built on piles and was not carried to rock as originally provided. This change was approved in writing by the joint board, and allowances were made for the change as provided in contract. Piers Nos. 2 and 3 were to be built on solid rock and were so built in pneumatic caissons. Piers Nos. 4 and 5 were built on piles sawed off as specified, but were built on grillages instead of directly on the piles. The latter was considered as a change, but no record was found of its approval by the joint board. As the engineer was dead and the members of the joint board probably did not know the difference between an open caisson and an open cofferdam, none of them had any recollection of the matter having been brought to their attention. The contractors were therefore in the position of having carried out the plans of the engineer in good faith, and because the latter did not get the written approval of the joint board, doubtless considering these changes a matter of detail, the contractors were charged with the responsibility for this lack of authority. It would therefore seem that more than an engineer's signature on plans and his written order to do work may be necessary to protect a contractor from trouble, both financial and otherwise, even though the work is done in good faith and to the satisfaction and acceptance of the engineer.

PERSONNEL.

The bridge was built under an act of the Massachusetts Legislature by a joint board of nine men, consisting of the Harbor and Land Commission, the Railroad Commission, and the County Commission of Bristol County. Mr. E. K. Turner was the engineer in charge of making plans and specifications, and Mr. R. M. Berrian was engineer of construction until his death, when Mr. Turner again took up the work and finished it, with Mr. R. F. Stoddard as resident engineer during the entire construction. J. P. Worcester & Co. were the engineers for the steelwork. The Holbrook, Cabot & Rollins Corporation, of Boston, had the general contract for the original bridge and for the reconstruction work.

Some Methods and Costs of Encasing Steel Structures in Concrete to Prevent Corrosion.

Two methods of encasing steel structures in concrete, namely encasement by pouring and encasement by cement gun, are discussed in one of the appendixes of the report of the Committee on Steel Structures of the American Railway Engineering Association. The report states:

(1) If the floor is protected by concrete encasement poured in place, the cost will be approximately 25 cts. per square foot for an envelope 3 ins. thick.

(2) Encasement 3 ins. thick placed by cement gun will cost approximately 23 cts. per square foot.

Specific samples of concrete encasement work are quoted as follows from W. F. Jordan, manager Grand Central Terminal Improvements, New York Central & Hudson River R. R., and from G. E. Tebbetts, Bridge Engineer, Kansas City Terminal Ry.

GRAND CENTRAL TERMINAL

The cement gun is being used at the Grand Central Terminal for fireproofing and protecting a part of the steel structure. The yard is in two stories, the upper tracks being supported on a steel structure with concrete jack arches. It was necessary to get the upper tracks in service at an early date, so the fireproofing of the exposed parts of the steel below the jack-arches was not done at the time the floor was built.

The lower parts of the beams, the girders and columns are now being fireproofed with the cement gun, using a minimum thickness of 2 ins.; the average thickness is from 2½ to 3 ins., as in the angles and around the stiffeners there is generally more than the minimum thickness.

The fireproofing is reinforced with a wire mesh, 1½x1½ ins. of No. 12 wires; this is attached to ¼-in. rods, which are bent around the steel and fastened to it.

The mixture has generally been 1 to 3, but in cool weather, and where the steel is subject to vibrations from the trains running on it, a 1 to 2 mixture is found to be more economical, as it is not as likely to drop off. It is necessary with this machine to use fine sand, as sand with pebbles in it clogs the hose; all of the sand, therefore, has to be carefully screened.

We find that a cubic foot of 1 to 3 mixture, when weighed in a box of 1 cu. ft. capacity after being moderately shaken down, weighs 93 lbs.; if this mixture is wet and applied with a trowel, after setting it will weigh 127 lbs. to the cubic foot, when shot through a cement gun onto a steel structure and set up, it weighs 144 lbs. per cubic foot. From this one gets an idea of the density of the fireproofing made with this apparatus.

In applying the mixture of sand and cement with the cement gun from 20 to 25 per cent of it is lost. Some bounces off as it strikes the structure, some is shot by the steel in working around the angles and to get a smooth surface the mason scrapes off the irregularities, and to get a good surface it is floated.

Fig. 1. Typical Concrete Encasement of Deck Plate Girder, Kansas City Terminal Ry.

The labor required to operate one machine is as follows: 1 foreman, 1 operator of the machine, 1 nozzleman, 2 masons for floating, 4 laborers screening, mixing and charging the machines. Carpenters are used when necessary to erect scaffolds.

One of these machines uses compressed air to the amount of 100 ft. of free air per minute at a pressure from 35 to 40 lbs. The hose through which the mixture is conveyed wears out quite rapidly and renewals amount to about $1 per day. We have averaged covering about 500 sq. ft. per day of the thickness mentioned above. This method would appear to give an excellent protection for the steel. The material is very dense and the method of application such that every inch of the structure is uniformly protected. The great thickness used in this work is due to the municipal laws requiring at least 2 ins. of fire protection.

KANSAS CITY TERMINALS.

On the new structures, which the Kansas City Terminal Railway Co. is building, the encasement is applied in the majority of cases by use of the cement gun. This machine consists essentially of a hopper into which the cementitous materials, made up of 1 part Portland cement to 3 parts dry screened sand, are placed; a hose connected to the bottom of the hopper, through which the mixture is forced by air pressure; a nozzle at the end of the hose, to which another hose supplying water is attached for hydrating the materials. At the end of the hose is a cylindrical nozzle having an annular ring at its base, to which the hose delivering the water is attached. This water is delivered inside the nozzle in the form of a fine spray, through which the materials from the gun pass. The nozzle is made of brass, and to prevent wear on the nozzle proper a rubber lining is used. This lining can be replaced whenever necessary.

Before adopting the cement gun, the claims of the company selling it were investigated and test panels were encased. The conclusion reached was that if the cost was not too great, it would solve the problem of encasement. Comparative estimates made are shown below:

Encasement by Pouring in Forms.—Encasement to be 3 in. in thickness; mixture to be 1:2:4 concrete; reinforcement, wire mesh and bars.

Stone, 1 cu. yd. at $1.25	$1.25
Unloading 1 cu. yd. at 20 cts.	.20
Loss in handling at 5 per cent	.07
Sand, ½ cu. yd. at 60 cts.	.30
Unloading ½ cu. yd. at 6c	.03
Loss in handling at 5 per cent	.02
Cement, 1¾ bbls. at $1.25	2.19
Unloading 1¾ bbls. at 5 cts.	.09
Loss in sacks at 5 per cent	.03
Total	$4.18
1 cu. yd. equal to 108 sq. ft., 3 in. thick.	
Cost of material per sq. ft.	$0.039
Forms, $1.63 B. M. at $0.050	.081
Mixing and placing at $5.40 per cu. yd.	.050
Insurance on payroll at 5 per cent	.003
	$0.173
Overhead and profit at 8 per cent + 15 per cent = 23 per cent	.040
Cost per sq. ft. of encasement = $0.216.	
Encasement per sq. ft.	$0.216
Mesh No. 3 at $0.06	.018
Bars, No. 5, at $0.03	.015
Total cost per sq. ft.	$0.249
Say, 25 cents per square foot.	

Encasement by Use of Cement Gun.—Encasement to be 3 ins. in thickness; mixture 1:3 mortar; reinforcement, wire mesh and bars. Average number of square feet covered in a day of 10 hours, 275 sq. ft. Loss due to gun work, 20 per cent. Loss due to handling sand, 30 per cent. Quantity of sand used in placing 275 sq. ft. 3 ins. thick, 4 cu. yds.

Sand, 4 cu. yds. at $0.60		$ 2.40
Unloading and screening 4 yds. at $2.51		1.00
Cement, 5½ bbls. at $1.25		6.88
Unloading 5½ bbls. at $0.15		.83
Loss in sacks at 5 per cent		.11
Water, per day		.15
Gasoline for compressor, 12 gals. at $0.15½		1.86
Oil waste and handling per day		.60
		$13.83
1 foreman, 10 hrs., at 37.5 cts.		$ 3.75
1 finisher, 10 hrs., at 35 cts.		4.50
1 nozzleman, 10 hrs., at 32.5 cts.		3.25
1 gunman, 10 hrs., at 30 cts.		3.00
2 laborers, 10 hours, at 22.5 cts.		4.50
1 boy, 10 hrs., at 12½ cts.		1.25
		$19.25
Repairs, etc., per day	$2.00	
Scaffolding for 275 sq. ft. at $0.15	4.13	
		6.13
Interest on gun, $3,000 at 5 per cent	$0.41	
Insurance on payroll at 5 per cent	.97	
		1.38
Total		$40.59
Overhead and profit 8 per cent and 25 per cent = 33 per cent		13.53
Cost of encasement		$54.12
Cost per sq. ft.		$0.1968
Mesh No. 3, per sq. ft. at $0.06		.018
Bars, No. 5, per sq. ft. at $0.03		.015
		$0.2298
Say, 23 cents per square foot.		

A comparison of the above shows a saving of 2 cts. per square foot in favor of the gun work over the poured encasement, and it might be stated that since this estimate was made we have received bids on actual work that check very closely with the above.

The steel work to be encased was designed with open holes 11-16 in. in diameter in webs, stiffeners and flanges, so that in placing and attaching the reinforcement there would be ample provision for rigid attachment to the structure. In attaching reinforcement to girder webs and other large surfaces, the bars were placed on small V-shaped iron saddles and wired through the webs to each other. On flanges the rods were run through steel eyebolts attached to the lower flange, the mesh being attached to the bars by wiring. At the junction of the concrete encasement with the floor, which is also of concrete, a splice was provided by use of mesh placed in the floor previously cast, this splice being four inches in width. Figure 1 shows typical method of attaching reinforcement to girders.

The steel girders were shipped from the shops with a shop coat of linseed oil, which was removed by the use of a caustic soda wash before encasement was started. All rust spots were removed with a wire brush.

Our experience has shown that the 1:3 mixture placed in the gun gives a resulting mortar of approximately 1:2½, this change being due to loss of sand. The sand must be nearly dry, the dryer the better, a mixture of coarse and fine grains giving better results with considerably less loss, than either the coarse or fine alone.

The sand must be screened as particles over ⅛ in. in diameter clog the gun and cause serious delays.

The compressor should be a machine of very ample capacity and an intermediate air storage tank is an advantage.

It was found that it was very difficult to encase the lower flange of a girder, especially so the lower face, and in our work we cast this portion in quite a few cases.

It was also a difficult proposition to get a good, clean job around stiffeners and sidewalk bracket members. On the brackets V-shaped forms were made and used as a backing for the gun work. As to finish, the appearance is fairly good, though far from the smooth, even lines of cast work and a great deal depends upon the finisher as to the final appearance.

Roads and Streets

Equipment and Methods for Maintaining Bituminous Surfaces and Bituminous Pavements.

The importance of thorough and continuous maintenance of bituminous pavements is impressing itself upon the minds of engineers engaged in highway work more forcibly every day. The tremendous increase in the square yardage of new construction of that type of pavement within the past few years has kept builders so busy with new methods and improvements in materials that better methods of maintenance have suffered somewhat from neglect. At the present time, however, many well constructed pavement surfaces are beginning to show those signs of neglect which, unfortunately, are characteristic of pavements in many sections of this country.

In order to carry on a system of continuous maintenance it is necessary to organize and equip small gangs which may be readily increased in size should occasion demand, and which, by working continuously, become expert in the details of their work. Mr. William H. Farrington, M. Am. Soc. C. E., in a paper published in Proc. Am. Soc. C. E., Vol. XL., pp. 157-163, discusses the organization, equipment and methods of carrying on this class of work and the plan outlined is given here as he presented it.

The maintenance of bituminous surfaces and bituminous pavements is divided naturally into three stages. The first includes what is usually classified as ordinary maintenance, such as patching, covering, etc. The second covers re-treatment of the surfaces and the renewal of the seal coats on the pavements; and the third includes the re-shaping or re-surfacing of roadways carrying bituminous surfaces, and the re-surfacing or reconstruction of the pavements.

It is assumed that maintenance work will usually be handled by a department with its own gangs and equipment, rather than by contract, except possibly in case of extensive re-surfacing.

Patching.—For patching, only a small gang is needed, and the equipment is simple. There should be one or two portable melting kettles, a small portable tool-house, and one or two small tents for the laborers, unless a van is provided.

The methods to be followed in patching the different kinds of surfaces and pavements are much the same. Generally, one should use the same kind of bituminous material for patching as was used in forming the surface, and ordinarily it is not advisable to select a much heavier grade, except with stone, slag, or chips, for filling depressions.

In the writer's opinion, only the lightest, or so-called dust-laying, oils or tars should be used without the addition of some grit; and, even with these, a light covering is probably economical. If applied without covering, however, patching is usually not advisable, as the cost of an additional application is small.

In patching an ordinary oil or tar surface, one should first attend to the depressions. Slight depressions can be filled with oil, or tar, and sand, gravel, or chips; but if they are deep, stone or slag should be added. The depressions should be cleaned, and the edges cut out if necessary, in order to allow a thorough bonding of the new and old material.

Although the best results are obtained by mixing the bituminous material and sand, or stone, etc., before applying, satisfactory results may be obtained without mixing, if the work is done carefully. In filling slight depressions, where the materials are not mixed before applying, a thin coating of oil or tar is spread over the bottom and covered with sand, gravel, or chips. If stone or slag is used with oil for filling deep depressions, the stone or slag is spread over the oil with which the bottom has been coated, then more oil is applied and covered with grit. If tar is used, the methods are similar, but more bituminous material is required.

After filling the depressions, any bare places, or places where the bituminous surface is practically worn through, should be coated, the surface first being cleaned and grit being spread over the oil or tar. In no case should much more than ½ gal. of bituminous material per square yard be used for coating the surface, as any surplus oil or tar will result eventually in the rolling of the surface and the formation of bunches.

Most of the so-called cold tars may be applied without heating, as may the lighter grades of the so-called cold oils, or road oils; but the heavier grades must be heated in order to give the best results in patching.

A much heavier oil or tar than that used in the surface is inadvisable for patching, yet a somewhat lighter grade may be used. Excellent results are obtained by patching a hot oil or tar blanket with the heaviest cold oil or tar, and with these there is much less chance for the formation of bunches than with the heavier materials.

If a pavement composed of stone and bituminous material is constructed properly, not much in the way of maintenance should be required for some time, but depressions will develop eventually, and some patching will be needed. In patching bituminous pavements, including those composed of sand and oil, the methods are similar to those for a bituminous surface, except that, with the sand and oil pavements, deep depressions should be filled with heavy oil and sand, properly heated and mixed before applying; or, if the heavy oil is not available, or there are no facilities for heating the sand, an oil similar to that used for surface work will give good results with slag or cinders, without mixing.

Covering.—On any bituminous surface, and especially one in the formation of which oil is used, more or less covering with sand, gravel, or chips will be required, either to take up free oil or tar, which will work to the surface for some time after it is applied, or later to prevent the picking up of the surface, which may soften under certain conditions. Covering is also partly a remedy for the slippery condition incidental to any bituminous surface during cold weather.

Re-Oiling or Re-Tarviating.—For re-treating a bituminous surface, practically the same equipment is required and the same methods should be followed as for the original treatment, except that usually less bituminous material is needed.

The life of a bituminous surface, or the length of time elapsing before re-treatment is necessary, will depend on local conditions, and the amount and kind of travel, etc. Where conditions are favorable for such treatment, however, and the travel, especially the horse-drawn travel, is not too heavy, a bituminous surface consisting of grit and about ½ gal. per square yard of the heaviest hot oil suitable for blanket work should last from three to five years without re-treatment, if patched occasionally and otherwise maintained properly. In re-treating such a surface, not more than ¼ gal. of oil per square yard should be used, unless practically all the original surface is gone; and it is not economical to delay re-treatment until this point is reached, as without the protection of the bituminous surface, the road metal will tend to ravel and the roadway to disintegrate.

A bituminous surface formed by the application of from 1/6 to ¼ gal. of light oil per square yard, with a small quantity of covering, will last at the best not more than one season, and it will probably be necessary to cover such a surface quite heavily in the fall in order to carry it through the winter and until another application is made the following year. Where from ⅜ to ½ gal. per square yard of the heaviest cold oil is used with grit, re-treatment with about the same quantity of oil will usually be necessary the second year; but, after that, the application of from 1/6 to ⅓ gal. per square yard per season should be sufficient, and an application may not be necessary each year, although if omitted the patching will naturally be increased.

If refined coal-gas tar is used, about ½ gal. per square yard is required during the first and second seasons. After that, with cold tar, a ¼- to ⅓-gal. application will usually be necessary after each season, although the yearly application may be omitted occasionally. With hot tar, ordinarily a ¼- to ½ gal application should be made every second year. Where unrefined water gas tar is applied, two ⅜-gal. applications are advisable each year, but, if only one ½ gal application is made, a considerable quantity of patching will be required.

As to equipment, for handling the lighter grades of bituminous materials, one or two distributor carts and a pump for transferring these materials from the tank-cars are required, and there should be a tool-house, also tents for the laborers, unless vans are provided.

For handling the heaviest grades of oil or tar used for surface work, the following plant is needed: One 10- or 12-ton steam roller, one portable heating boiler, one steam pump, two steel tank-wagons, one pressure distributor (if mounted separately, or two if attached to the tank wagons), one or two watering carts, one 2-horse sweeper, one portable tool-house, and two large tents or vans for laborers.

As to distributors, for handling the road oils or the corresponding grades of tars, a gravity machine may be used with good results, if the bituminous material is broomed after it is applied; but the cost of distribution will be less and the results more satisfactory if a pressure distributor is used. For this work, a distributor cart with a pump mounted on a frame back of the tank and connected by chain with a large sprocket bolted to one of the rear wheels is most satisfactory. Under favorable conditions, good results have been obtained with gravity machines in distributing even the heaviest grades of bituminous materials for surface work; but the chances of failure, if the conditions are not favorable, are such that they should always be applied by pressure distributors if possible. The distributor just described, or one with a gas-engine pump mounted either on the tank-wagon or on separate gear, may be used, but one operated by steam will give the best results. One advantage of the steam distributor is the possibility of cleaning out the pipes and nozzles by blowing steam through them. Of the steam distributors, there seems to be little choice between the one using the direct pressure of steam in the tank for distributing and the pump type, each having its advantages.

To facilitate covering, the sand or other material should be piled in advance on the side of the road.

Before commencing to distribute the bituminous material, the surface should be cleaned with a horse sweeper, and with hand-brooms, if necessary. Any depressions should be filled and the surface patched as needed.

Where the heavier grades of oil or tar are used, it is well to water the old surface before distributing the new bituminous material, but watering is not necessary with the lighter grades unless the surface is unusually dry.

It is advisable to cover the bituminous material lightly at first, applying more grit as needed for though some bituminous material will take up only enough grit to form a good mixture, without reference to the quantity put on, in other cases, if there is surplus covering, the oil or tar will continue to work up until the mixture is weakened to such an extent that the best results cannot be obtained.

Seal Coats.—Although a seal coat or wearing coat may not in all cases be necessary on pavements composed of broken stone and bituminous material, the writer believes that it is economical and should usually be applied, either in connection with construction, or later, except possibly where asphalt is used in the pavement. The equipment and methods for applying or renewing such coats are similar to those for the re-treatment of the surface; but, unless asphalt is used, the bituminous material for the seal coat should be of a lighter grade than that incorporated in the pavement. Oil may be used with good results on a tar pavement, and is advantageous in some cases, in that it is less slippery than tar and also acts to some extent as a dust absorbent.

Re-Shaping or Re-Surfacing.—Although a surface consisting of grit and suitable bituminous material will carry ordinary travel and relieve the road metal of much of the wear, it alone is not a permanent form of treatment. As a result of the gradual wear of the road metal, and because repeated treatment and patching with bituminous material will cause the formation of bunches and an uneven surface, re-shaping or re-surfacing of the roadway will be necessary eventually.

When this stage is reached it is always well to consider a more permanent form of pavement; for it is an established fact that a bituminous surface will not carry a large amount of horse-drawn travel, and will fail under certain conditions which the writer believes are not as yet clearly understood; and, although a bituminous surface may have been satisfactory in a certain locality in the past, the conditions may change or the travel increase largely.

It is assumed, however, that there will not always be money available for the construction of a bituminous pavement, in which case the alternative is the re-shaping or re-surfacing of the roadway and the application of another bituminous surface.

The equipment needed for re-shaping or re-surfacing work, to be done in connection with re-oiling or re-tarviating, is the same as that for re-treatment, except that two steam rollers will be needed, also a straight-tooth and a spring-tooth harrow.

If stone is to be crushed, there should be a portable crushing outfit.

In re-shaping or re-surfacing a roadway with a bituminous surface, the old surface should first be thoroughly broken up with the roller picks or a scarifier and removed with stone forks.

The loosening of the road metal, partly accomplished by the roller picks or scarifier used in breaking up the surface, should be completed with a straight-tooth harrow, the use of which also tends to shake the worn out material to the bottom. This should be followed by a spring-tooth harrow, which, because of the curved teeth, will bring the stone or larger particles of road metal to the top. The surface should be shaped with rakes, any new stone or other road metal needed should be put on, and the road watered and thoroughly rolled. If any considerable quantity of new stone, etc., is used, the voids should be partly filled with sand, stone screenings, or chips; but, if only a small quantity of new road metal is put on, there will usually be sufficient fine material in the road to bond it.

Either a macadam binder or the heaviest road oil may be used, but, ordinarily, the former will give the best results. About ½ gal. of oil per square yard should be used in two applications, with a covering of grit after each.

Ordinarily, it is not advisable to use tar for such re-shaping or re-surfacing work, unless a bituminous pavement is to be formed by penetration. In this case, usually more new stone or other road metal is needed, and the voids for about 2 ins. below the surface should not be filled with fine material. For grouting 1¼ gal. of heavy tar should be used, and a seal coat should be formed by the application of about ½ gal. of a lighter tar with grit; or, if preferred, asphalt may be used instead of tar.

The foregoing description of maintenance work applies to a bituminous surface on a roadway of broken stone, slag, or screened gravel, rather than to such a surface on unscreened gravel; but though the methods for handling a surface on the latter form of roadway are somewhat different, mention can only be made of the fact that although, for the first application, one should use a fairly light grade of bituminous material which will penetrate somewhat below the road surface and thus form a real bond between the bituminous material and the gravel, heavier grades of oil or tar may be used, with good results, for later treatment.

Re-Surfacing of Bituminous Pavement.—In a discussion on maintenance, it does not seem advisable to devote much space to the re-surfacing of bituminous pavements, because such work is probably better classified as reconstruction than maintenance, but attention is called to the fact that the life of pavements composed of broken stone and bituminous material is as yet problematical on roads outside of cities and large centers, because such pavements have been in use, under these conditions, only a few years.

Storage and Repairs.—Incidental to a consideration of equipment is the question of proper care and up-keep of the outfits. There should be at headquarters, a suitable storage place, with sufficient equipment and proper facilities for making ordinary repairs. It is not only advantageous to have the repairs made by or under the supervision of those responsible for the apparatus during the working season, but, by having the work done by men from the regular force, it is possible to keep the most competent men busy during the winter.

If the amount of work done warrants the employment of a storekeeper, there should be kept at all times at the storage place a sufficient stock of tools, etc., to supply the needs of all the gangs.

Reports and Accounts.—It is not possible, in the time at the writer's disposal, to consider in detail the question of the reports, records, and accounts which should be kept on maintenance work. The writer believes, however, that although a consideration of cost is not of especial value in a discussion covering work in widely separated localities, a comparison of the cost of work in different localities in one section is valuable, and is necessary in order to determine what equipment and methods are most economical, and also whether the work is handled efficiently.

Daily reports giving details of the labor cost should be submitted by the foreman, and a record of these, as well as of bills for materials, supplies, etc., should be kept in such a way that the cost of the different kinds of work can be easily ascertained at any time.

Methods and Cost of Laying 5,689 Sq. Yds. of Vitrified Brick Pavement at Carlisle, Pa.

Contributed by John C. Hiteshew, City Engineer, Carlisle, Pa.

During the past season 5,689 sq. yds. of vitrified brick pavement were laid by the street department of Carlisle, Pa., under the supervision of the writer. The section paved comprised two blocks on High St. and one block on Hanover St. On Hanover St. the paving extended over the full width. The Valley Railways Co., whose tracks occupy the center of the street, in accordance with the term of their franchise, paved the portion taken up by their right-of-way. On High St. the Cumberland Valley R. R. built a concrete curb along their tracks.

Concrete curbs were used throughout and were constructed by contract. The curb along the tracks of the Cumberland Valley R. R was 6x18 ins. in dimension, set at a distance of 2 ft. from the inside of curb to rail. The concrete was a 1:2:3 mix with a 1:1 finish coat and was constructed at a contract price of 35 cts. per linear foot. The curb for the borough (1,200 ft.) was of a 1:2:4 mix with a 1:2 finish coat and was constructed by the same contractor at a contract price of 28 cts. per linear foot.

The intersection of High St. and Hanover St. being a summit, the paving at the street crossings was made flush with the sidewalls, as shown in Fig. 1. The same construction was used at alley crossings. This method of treating crossings is similar to that used at Atlantic City, N. J.

The pavement was laid in short sections, the sections being so alternated that traffic was not seriously interrupted at any time. The old macadam pavement with which the street was formerly paved was used as a base for the brick paving. In grading the base the old macadam was spiked up with a 13-ton Kelly-Springfield roller, shaped and thoroughly dry rolled. In shaping the base spikes were set to grade at 7 ft. intervals across the street and the final shaping done by hand picking. After rolling new spikes were set which supported the 2 x 8-in. x 16-ft. guides for the striking template. The cushion coat of crushed stone screenings was placed, and struck off to a depth of 1½ ins. by means of a short striking template drawn by two men, as illustrated in Fig. 3. The cushion was rolled with a 500-lb. roller drawn by hand.

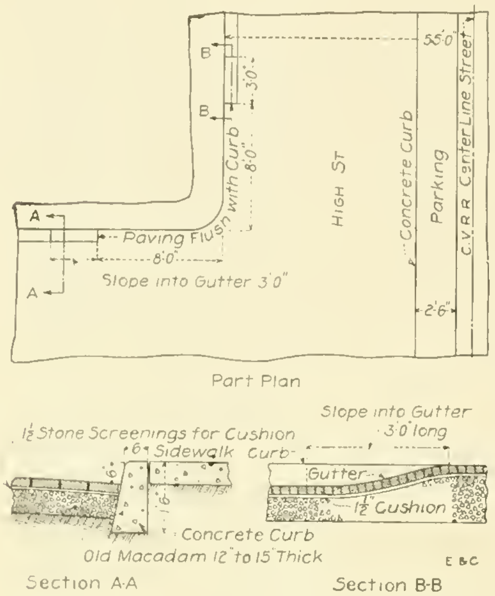


Fig. 1. Part Plan and Sections Showing Details of Street Crossing Having Pavement Flush With Sidewalk.

The brick were laid by one foreman and two men, one of the men breaking half-brick to fill in ends of courses. Usually there were three men carrying to each brick layer or an average of 7,000 brick were laid by each man per day. Vitrified shale paving brick were used. Rolling was done with a 5-ton horse roller drawn by twelve men. The joints were grouted with a 1:1½ cement grout. The pitch expansion joints were filled after the grout had set sufficiently to allow the removal of the expansion boards. The green pavement was covered with ½-in. of sand and kept damp for several days. Traffic was turned on at the end of five days.

Fig. 2. Method of Setting Concrete Curb Along Railroad Tracks.

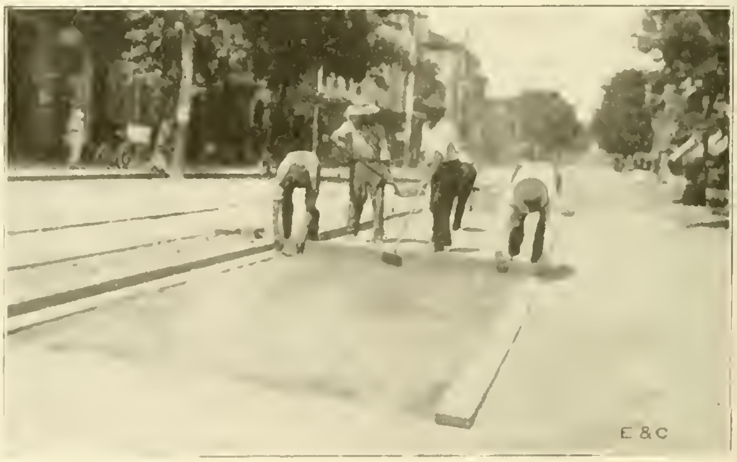


Fig. 3. Method of Using Short Striking Template.

TABLE I.—COST OF BRICK PAVING AT CARLISLE, PA.

Item—	Cost per sq. yd.	Total cost.
Unloading, hauling & piling bricks	$0.065	$369.78
Grading & rolling macadam base	.091	517.70
Stone cushion and rolling with 500-lb. roller	.052	295.83
227,560 brick at $21 per M	.840	4,778.76
Laying and rolling brick	.034	193.43
Cement grout filler	.068	386.85
Expansion joint	.010	56.89
Totals	$1.16	$6,599.24
Total number of sq. yds		5,689
Cost per sq. yd		$1.16

Note.—An average of 7,000 brick per man per day were laid.

TABLE II.—COST OF LABOR AND MATERIALS FOR BRICK PAVING AT CARLISLE, PA.

Item—	Rate.
Foreman, per hour	$ 0.25
Laborers, per hour	0.16
Roller engineer, per hour	0.25
Cement, per bbl. (net)	1.32
Sand, per ton	1.50
Brick, per M	21.00

Factors Limiting the Selection of Materials and of Methods in Highway Construction.

There is no feature connected with the economic design of highways which calls for more careful study and consideration on the part of the engineer than the selection of the type of surfacing to be used. A sufficient number of data must be gathered so that the choice of wearing surface and its attendant details such as crown, maximum curvature, width, etc., may be based upon facts and figures and not upon the personal preference of the engineer or board of commissioners. While this is always the effort of the conscientious road official, it is often a difficult matter to determine just what factors enter into the question and the relative importance of these factors. In this connection, it is well to remember that an economic surfacing is not merely a function of the first cost of the improvement, but is dependent upon a variety of complicated conditions among which may be mentioned: the character of the road and supporting country, traffic, materials available, legal considerations, and climatic conditions.

As a means of approaching their question, Mr. P. E. Green, M. Am. Soc. C. E., has evolved a diagrammatic outline of factors in road work which is interesting and new in some of its features. The diagram and Mr. Green's discussion are given here as presented by him in a paper entitled: Factors Limiting the Selection of Materials and Methods in

Highway Construction, published in Vol. XL Proc. Am. Soc. C. E., pp. 441-446.

The subject of economic construction may be logically divided into two interlocking general sections, namely: "The Selection of Materials," and "Methods of Construction." Each of these main headings may be further divided into numerous and important parts. All these parts should be taken into consideration by any public authority, such as a city, county, or state, when it is proposed to enter on a general scheme of highway improvement such as is so common at this time. In this way it is not difficult to make a logical selection of the various materials and methods involved in the construction of a system of streets or highways; and the scheme or plan of future construction may be laid out. This does not mean that only one material shall necessarily be selected for any one street or highway. It simply means that the class or type of material shall be selected, and that choice must ultimately be made between several more or less suitable products.

A diagrammatic outline of such a general scheme covering the subject of this discussion is herewith submitted. It is not claimed that it is particularly novel or complete, but is presented for discussion and improvement. Although intended primarily for country highways, it may be used with equal facility for city streets.

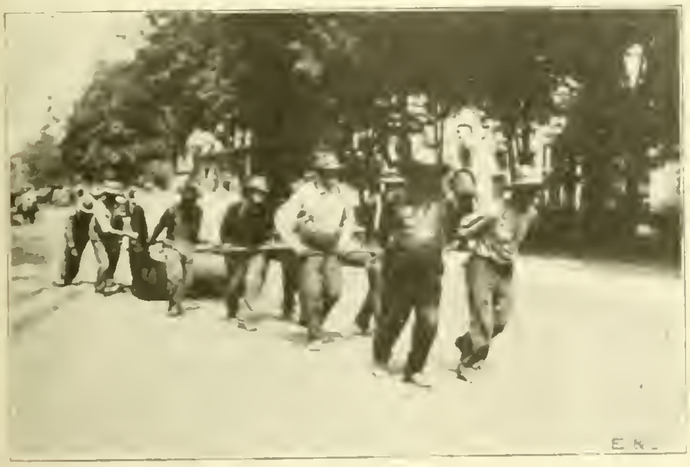


Fig. 4. Five-Ton Roller, Drawn by Twelve Men, Rolling Pavement Prior to Grouting.

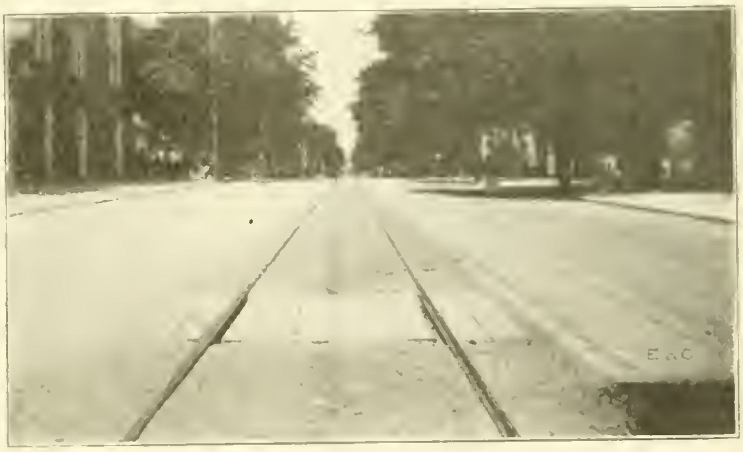


Fig. 5. Finished Surface of Brick Pavement at Carlisle, Pa. No Paving Between Car Tracks.

The scientific selection of the material to be used in the construction of a highway depends on all the items named in the diagram. The diagram itself looks somewhat complicated, but its use is very simple. Before commencing any large scheme of highway improvement, the outline may be studied with reference to materials, methods, etc., and simple tables prepared. Too often some essentials are not considered seriously or are forgotten, and for this reason much money, time, and labor are wasted. From time to time, somewhat similar schemes have been advanced, but frequently they deal with only one essential, such as traffic, to the exclusion of other important elements. For example, in addition to traffic, the character of a city highway should depend on the layout of the city, its topography, population, business, education, and wealth; and, further, the location of the city as regards the availability of standard materials, such as brick, wood block, crushed stone, gravel, sand, tar, asphalt, etc., etc., is vitally important.

Considering the problem of a country highway system, many entirely different factors enter into the selection of the material, such as vicinity of cities, large or small; general population and wealth of the surrounding country; whether or not the highway is to be a trunk line for through traffic, and if so, whether such traffic is light, high-speed traffic, or heavy, low-speed, commercial traffic.

If the highway is a secondary one, as regards the through traffic, it still may be a trunk line as regards its commercial traffic, such as a road leading from a trucking district to any one of our large cities. Such a highway may be comparatively a short one, yet the character of its traffic actually makes it a trunk line.

The least important class of country roads is generally referred to as "local roads." These are exactly what the name signifies, and whether or not they are to be improved depends on the wealth and education of the surrounding country and its inhabitants. Such roads are generally paid for only by local taxation.

The effect of "climate" is more important than is often considered. Some pavements, such as the various bituminous pavements, concrete, etc., are very much affected by the temperature. Thus a surface mixture of asphalt suitable for Chicago is not suitable for New Orleans; and concrete as a wearing surface should do better in Houston than Buffalo.

Three zones may be recognized: Northern, embracing the New England States, New York, Pennsylvania, Ohio, Illinois, etc.; the Intermediate, such as Virginia, Kentucky, Tennessee, Arkansas, etc.; and Southern, such as Alabama, Mississippi, and Texas, which have little winter. In the Northern States, the selection must be of materials not effected by extremes of temperature, freezing and thawing. For the Intermediate, it is not necessary for the materials to withstand great variations in temperature. For the Southern, materials which might utterly fail in the North because of inability to withstand extreme cold or great variations in temperature may be very satisfactory. Furthermore, the exceedingly heavy and continuous rains following long hot, dry spells in the southern sections must be taken into account.

A great deal of money might often be saved by considering carefully the various classes of paving material, which may be suitable for the same purposes and yet may vary widely in cost in the different parts of the country. Thus, wood block pavement may be a very reasonable one for the business streets of small cities in regions close to the sources of supply, but for other cities, similar in size, but not so situated, it would be a grave mistake to use it, because of its prohibitive cost. This same wood block pavement has a characteristic in common with sheet asphalt: it must receive a certain minimum traffic or it may fail. On a wide roadway, having only a traffic along the center, the blocks on either side of this used strip will dry up under the sun and swell and "blow up" if a rainy fall follows a hot dry summer.

Further, such a material as trap rock, [illegible] good macadam [illegible] in New York, probably better, as far as wear is concerned, than any of the limestone used in the West or South, would be impossible for these Southern sections. Yet the writer has seen the [illegible] for roads hundreds of miles away from granite quarries, which called for granite top macadam, though local limestone or [illegible] would wear nearly as well. Again, there are sections of the country where the high cost of cement and the comparatively low cost of good stone or gravel make a concrete base for a pavement quite expensive, when a crushed stone or a natural gravel rolled base would cost less and be nearly, if not entirely, as durable. Of late years, too little attention has been paid to the

DIAGRAMMATIC OUTLINE OF "FACTORS" IN ROAD WORK.

Highways Under Consideration

- I. Selection of materials.
  - 1. Character of road (city or country)
    - Trunk
      - Through traffic road—commercial or passenger, or both
      - Commercial road, but not necessarily through traffic.
    - Secondary
      - Connecting important centers to trunk lines.
    - Local
  - 2. Traffic
    - Much light fast passenger, or
    - Heavy hauling, or
    - Only local hauling
    - Roadway width required
  - 3. Climate
    - Northern
    - Intermediate
    - Southern
  - 4. Character of supporting country
    - Property values and benefit to be derived
    - Vicinity of large cities
    - Topography, drainage
    - Character of business using road—[illegible] consideration.
  - 5. Materials
    - Class
      - Superior
      - Inferior
      - [illegible] long wearing, which [illegible] wear, crown, dust, sanitation, etc., etc.
    - Cost—maintenance charges.
    - Character and appearance
    - Ease of handling materials
    - Relation of material to roadway width
- II. Methods of construction
  - 1. Laws and legal organizations governing.
  - 2. Location of railways with respect to improvements—railway rates
  - 3. Existing good roads as affecting haul—wagon or truck
  - 4. Availability of labor.

macadam base as a foundation for a durable pavement. The public has been educated to think that concrete is the only possible foundation. The situation is similar to that of the septic tank in sewage disposal. Many public officials have heard and believe that such a tank turns sewage into drinking water.

In those parts of the country in which there has been much road work and improvement, it is much easier to get competent foremen and good labor than in sections where little, if any, of such work has been done. The result of good workmanship is very apparent. The local officials, also, are better educated in these matters and more inclined to give an engineer sway.

It should be remembered, also, that the character of the highways, whether city or country, must depend largely on the ability of the taxpayers to pay for them. A sparsely settled region may be very desirous of having good roads and be willing to be taxed for their construction, within reason, but Wisconsin, for instance, is far less able to pay $10,000 per mile for highways than is Massachusetts. The road mileage depends only to a limited degree on the population.

There can be little question of the benefit derived from the construction of good highways, but the degree of immediate benefit should have much to do with the selection of the material for such improvement. It might be, and frequently appears, that the benefits in the long run from a thoroughly high-class permanent construction will, theoretically, far exceed those derived from any cheaper construction. Because of the large immediate benefit received from the cheaper construction, however, and because the taxpayers are unable to pay for the more expensive construction, it may be much better finance to put in the cheaper highway, and thus not overburden the community. Future development and growth of wealth may provide equitably for the construction of a better improvement. It is a real recognition of this principle, not altogether possessed by some engineers, that has justified in many cases the voting of long-term bonds for short-lived improvements. Such a principle must be used with caution, however. It cannot be defended in New York or Illinois, but certainly can in Texas and Oklahoma.

In addition to the various points noted thus far, the esthetic features of the wearing surface of a highway should not be forgotten. In many cases, this feature should be made a very large factor in the selection of the proper material. Thus, a smooth, well built brick pavement might be nearly as noiseless as one of bituminous concrete, and might have a much less maintenance charge and be more sanitary; but, along a scenic highway where the adjoining country is very highly developed esthetically, such as in the vicinity of a large city where there are many great estates, it is quite probable that the bituminous concrete pavement would be considered more suitable, pleasing, and satisfactory by those who pay for it. Such a road would be used largely by automobilists and comparatively little by truck gardeners. If this highway were lined by truck farms, however, could there be any doubt that the selection, if made between the two, should be brick, especially as it has been demonstrated that the bituminous concrete does not withstand iron-tired traffic? This selection, moreover, would be the correct one, even if the pleasure traffic was very large.

One of the points which it is believed has been neglected in the past is the relation of traffic to roadway width, especially where traffic is dense. It seems to the writer that it is possible to establish a logical, if empirical, formula which would be a considerable help at times in the solution of this vexatious problem. Some years ago, in preparing a report on a popular scenic highway near Chicago, a traffic census was taken at various points, and in addition statistics on other popular park highways in Chicago were obtained. These were plotted, together with the various existing roadway widths, and several curves tried on the same sheet. In this manner, it was finally decided that a parabolic curve having the equation:

$$Y = \sqrt{\frac{X}{6} + 16}.$$

would express the desired relation. In this equation, Y = the roadway width, in feet, and X = the number of vehicles.

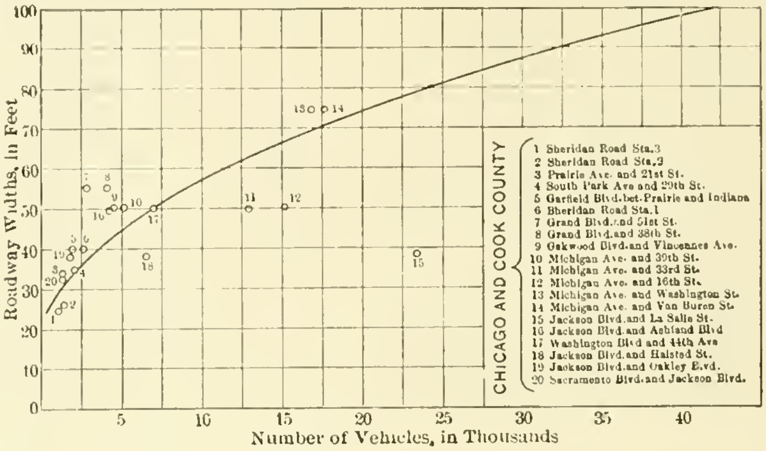


Fig. 1. Curve Showing Relation Between Width of Roadway and Probable Traffic for Several Streets in Chicago, Illinois.

Thus, if it is determined from a traffic census that within 5 or 10 years a highway will have a probable traffic of 2,000 vehicles, the formula would give a roadway width of 31 ft., as shown on Fig. 1.

The diagram, or formula, is of course faulty as it takes into account only two factors, but it has been a help in arriving at conclusions at times, and is offered for what it is worth.

Attention should be called to the fact that because it is decided that the roadway is to be of a certain width, say 34 ft., it does not follow that the entire road should be paved with an expensive material. Cuyahoga County, Ohio (Cleveland), has hundreds of miles of roads improved on one side with brick and on the other with plain macadam or gravel. The empty wagon and light horse traffic prefer the latter. Winona County, Minnesota, has paved the central 8 ft. of some of its highways with concrete, and provided wide macadam shoulders.

Sewerage

A Discussion of the Present Status of Water Supply and Sewage Disposal Conditions in Chicago.

The committee on drainage of the Chicago Real Estate Board made a report on January 7, 1914, covering the present condition of the water supply and sewage disposal of the city. The report criticised, unfavorably, some of the existing conditions, and a reply to it was prepared by Langdon Pearse, Engineer in Charge of the Sanitary Division, and submitted to the engineering committee of the Sanitary District of Chicago on January 29, 1914. The report by Mr. Pearse was published, however, but a short time ago and we are now presenting the substance of the two reports to our readers. The Harbor and River Improvement Committee of the Chicago Real Estate Board also made a report, dated February 4, 1914, in furtherance of the suggestions made by the Drainage Committee previously mentioned. This report contemplated the securing of outside expert advice relative to the sanitary situation in Chicago as will be hereafter more fully explained. In a further report by the Harbor and River Improvement Committee the engagement of three experts of international reputation was announced and the present article also covers this action fully. Since the critical point in the situation here discussed is the continued diversion of water from Lake Michigan for the purpose of diluting the city's sewage the problem is one of more than local interest. The asserted lowering of lake levels with the attendant effect upon shipping has made the problem one of concern to Canada as well as this country. The amount of water Chicago can divert for this purpose is now the subject of a suit in the federal courts as herein explained.

REPORT OF DRAINAGE COMMITTEE OF REAL ESTATE BOARD.

The members of this committee are Messrs. John W. Alvord, Henry H. Walker and Hyde W. Perce. This committee, in its report mentioned, assigned numbers to the various points raised and as these numbers were also adhered to by Mr. Pearse in his reply we are using them here. The various points made, both in the report of the committee and in Mr. Pearse's reply, can be here given only in substance but it is our purpose to present the essentials of both arguments as briefly as possible.

Taking up now the points raised by the drainage committee of the Real Estate Board we have:

(1) The condition of the city water supply during the past few months has been unsatisfactory in point of clarity and cleanliness.

(2) There has been a distinct rise in the typhoid death rate during the past year and this causes suspicion to be first cast upon the water supply as the cause.

(3) Health Commissioner Young states that Chicago suffers 17,000 deaths annually from preventable diseases. It is apparent that a considerable proportion of these preventable diseases may be caused directly or indirectly by an impure water supply.

(4) The city council has been stirred by these conditions to investigate the subject, and an agitation has been started to prevent promiscuous dumping of impure materials in the lake.

(5) Trustee Sullivan of the Chicago Drainage Board in reviewing the work of the Sanitary District, stated on Jan. 1, 1914, that 121,000 cu. yds. of septic sludge were removed from Bubbly Creek, in the stock yards district, during 1913, and dumped behind bulk-

heads at the lake shore at the foot of 12th St.

(6) In his report of Nov. 25, 1910, Mr. Wisner, Chief Engineer of the Sanitary District, in collaboration with Mr. C. D. Hill and Dr. W. A. Evans and others, demonstrated the following conclusions:

1st. The present drainage flow is insufficient to purify the sewage which now reaches it.

2d. Large quantities of filth produced in the stock yards district, and in the North Branch and West Fork of the Chicago River, are not brought within influence of the drainage canal flow and constitute a growing menace.

3d. The legal capacity required in the channel, 20,000 cu. ft. per minute per 100,000 population, is an insufficient ratio for effective dilution

4th. By a series of chemical tests the purifying capacity of the channel flow is shown to have been for some years past exhausted, especially in summer time by reason of the depletion of its dissolved oxygen.

5th Steps should be immediately taken for additional and further relief in the near future by artificial purification in aid of the channel, which additional works will cost the city a probable further expenditure of from twenty to twenty-five million dollars.

(7) The federal government has never authorized withdrawal from the Great Lakes at a rate of more than 4,167 cu. ft. per second of dilution water, and the State law creating the Sanitary District explicitly requires the withdrawal of 20,000 cu. ft. per minute per 100,000 population, which requirement at the present time cannot be much less than 8,000 cu. ft. per second, or about double the amount permitted by the federal government

(8) At this time and for some years past, the Sanitary District has actually withdrawn from Lake Michigan, at most times, between 7,000 and 8,000 cu. ft. per second, or very nearly the entire requirements of the State law, all in spite of the fact that this is in violation of the requirements of the general government.

(9) The mentioned report of Mr. Wisner is conclusive that not even this amount of flow is sufficient to effectively purify the sewage actually reaching the main channel in summer, while the sewage from the important areas named cannot at such seasons be brought into contact with the flow of the main channel until putrefaction is completed.

(10) The Sanitary District has brought suit against the general government in the hope of obtaining the authorization for a flow of 10,000 cu. ft per second, which amount was adopted by the International Waterway Commission as all that could properly be allowed the City of Chicago for its sanitary needs, but it is evident that even should the Sanitary District of Chicago be successful in this suit, and could compel the general government to allow 10,000 cu. ft. per second to be withdrawn, this amount of flow would only slightly exceed the amount which the Sanitary District is now withdrawing, and which amount has been declared insufficient for the present purpose of purification, by the Chief Engineer of the Sanitary District

Although the canal has been in operation only 13 years, it has reached the limits of it capacity for the present population. The matter has been further complicated by extending the District limits so as to give relief, through the Sag Canal to large areas to the south, and, through the Evanston channel, to large areas to the north The main channel was not designed to provide for these areas, and the International Waterway Commission declared that these added territories could be suitably and economically cared for by methods of sewage treatment.

The committee of the Real Estate Board concluded its report by stating that the time is ripe for the appointment of a board of experts, not previously connected with Chicago sanitary matters, to study present conditions and recommend a policy for the future sanitary protection of the city.

REPLY TO FOREGOING REPORT AS MADE BY LANGDON PEARSE.

Mr Langdon Pearse, Division Engineer, in charge of the Sanitary Division of the Sanitary District of Chicago, made a report to the chairman of the Engineering Committee of the Sanitary District in which he covered the present sanitary situation in Chicago, with particular reference to the statements published in the report of the drainage committee of the Chicago Real Estate Board, the gist of which report is above given. Mr. Pearse took up the matters contained in the committee report item by item and the substance of his report follows:

(1) The condition of the water supplied to Chicago during the past few months has been hygienically good. Tests show that the number of bacteria is low. The indications are that the water has been safe for people to drink.

During the last few months a storm occurred on Lake Michigan of unusual severity. The result was that the lake water was thoroughly agitated, the bottom of the lake being stirred up by wave action, thus making the water appear turbid as it was drawn from the faucets. This condition does not occur continuously, but only under such extraordinary storm action.

(2) The rise in the typhoid fever death rate in 1913 has been very slight, and of the deaths from preventable disease so-called, about 15,000 in the City of Chicago, only 246 were from typhoid fever, approximately 1.7 per cent of the so-called preventable deaths. This would be at the rate of a little less than 10 per 100,000, which is a lower death rate from typhoid fever than in most of the cities in the United States with a population of over 100,000. In 1912 only 4 cities out of 51 had a lower typhoid fever death rate than Chicago.

The slight increase in the typhoid death rate over that of the previous year has been accounted for by the Health Department as being due to a certain milk infection and to a "typhoid-carrier" who came from St. Louis, and was located in a down town restaurant, working as a waitress.

The analysis of the water supply shows that hygienically it was better in 1913 than it was in 1912

(3) The committee report gives the impression that a very large proportion of the 15,000 deaths from diseases said to be preventable may be traced to water, directly or indirectly. In 1913, however, only 246 deaths were reported from typhoid Of the other deaths, 3,253 were from diarrhoeal diseases of children under two years old, and of the total number of 35,291 deaths in the course of the year, 10,500 were of children five years and under. In 1913 preventable diseases claimed 15,000 victims, but in this list of preventable diseases are included such diseases as tuberculosis, pneumonia, diarrhoeal diseases of infants under two years, influenza, and whooping cough. Adding the actual number of deaths in 1913 from these causes we arrive at 12,153 deaths, and if to this we add diphtheria, croup, scarlet fever, measles, and smallpox, we make a total of 14,317. An inspection of these diseases and the number of deaths shows that the proportion of deaths due to causes commonly recognized as water borne diseases is very small It is also clearly apparent that in order to reduce the deaths from preventable diseases that sanitation must tend in other directions than the water supply.

(4) While the city council has investigated the conditions outlined and has frowned upon promiscuous dumping of impure material in the lake, the council apparently does not differentiate between impure water and muddy water, nor does it understand relation between dumping and turbidity Present laws are sufficient to control the dumping situation.

(5) Sanitary precautions were taken in dumping behind the closed breakwater at Twelfth Street the material taken from Bubbly Creek. Inspection and bacterial tests showed that such dumping exerted no practical influence on quality of water taken in at the supply cribs It was undesirable to carry this material eight miles out into the lake because the scows would have to pass by the crib at the mouth of the river, nor could the material be dumped elsewhere at reasonable cost.

(6) The exhaustive report to which the committee refers as of date of Nov. 25, 1910, was actually of date of Oct. 12, 1911. Mr Pearse's reply to each of the five subheadings under item 6 of the committee's report follows:

1st. Mr. Wisner, in his conclusions, says that the present scheme of sewage disposal by dilution has been a pronounced success, and that the canal for twelve years has furnished a most efficient and economical method of disposal of the sewage of the District.

2nd. Reference is made to large quantities of filth produced in the Stock Yards district and in the North branch. Mr. Wisner called attention to these nuisances, and stated that the conditions can be materially improved by the removal of the settling suspended matter.

3rd. The committee states that the legal capacity required in the channel, viz., 200 cu. ft. per minute per 1,000 population, is an insufficient ratio for effective dilution in Chicago. The ratio stated is fixed by law as the minimum dilution, and is fixed as a minimum dilution on the basis of domestic wastes only, on the assumption that industrial wastes will be removed, or so treated as not to require oxygen. Under the provisions of the law, the ratio therein stated is adequate to handle the domestic sewage.

4th. No reference is made by the committee to the fact that the exhaustion of the oxygen occurs many miles from the city, and that oxygen required to maintain the liquid under most favorable conditions is put back into the liquid at Lockport by passage over dams.

5th The committee states that it was the conclusion of the Chief Engineer that steps should be immediately taken for additional and further relief in the near future by artificial purification in aid of the channel, which additional works will cost a probable further expenditure of twenty to twenty-five million dollars. The Chief Engineer, however, clearly stated in his report that the steps to be taken were contingent upon the assumption of 10,000 cu ft per second as the diversion from Lake Michigan No one could tell at that time (or even today) what the diversion will be, as the matter is still in the federal courts These suggestions were made for further study and were not entirely based on the relief of the channel as such, but were to aid in clearing up local nuisances in places which the conditions of dilution do not reach.

(7) The committee would give the impression that the federal government has never authorized withdrawal from the lake at the rate of more than 4,167 cu. ft per second The Sanitary District, on the contrary, has never recognized the right of the Federal government to fix the total diversion If the committee had stated that the permit was only for the withdrawal of 4,167 cu. ft per second through the Chicago River, the truth would have been stated. For several years the Sanitary District has been drawing more water than that from the lake, at times up to 10,000 cu. ft per second, nor has it ever concealed this. The federal government at present is seeking to enjoin the District from using more than 4,167 cu. ft. per second total diversion from the lake If this contention is enforced, it will impress upon the community the immediate expenditure of over $10,000,000 to merely duplicate works now in service, without clear efficiency in protecting the water supply of the city and including cost of operation the damage to the taxpayers will exceed $200,000,000.

(8) The committee does not state its source of information concerning the withdrawal of water from the lake, nor where it learned that the withdrawal was a violation of the requirements of the general government The facts are the Sanitary District has never stip-

ulated or agreed with the federal government on the total diversion. It has always been understood by the government officials and others who were posted that the capacity of the channel of the District was and is 14,000 cu. ft. per second. In none of the published reports has mention ever been made of a diverison of less than 10,000 cu. ft. per second. Hence, there is no violation of the requirements of the federal government, because no requirement has ever existed, except in so far as navigation is concerned in a small part of the Chicago River. Water can be pumped in otherwise, through Thirty-ninth street and the South Fork. The Sanitary District was organized and the works now existing planned and built with full knowledge on all sides of the capacities of the channel and works and the intentions of the District. It was not until a few days before the opening of the completed channel in January, 1900, that any authority was assumed by the government, even as regards the question of velocities in the Chicago River, and then the authority assumed by the government was only over a diversion of 4,167 cu. ft. per second from the standpoint of velocities in a section which had not then been improved. The Sanitary District has now spent over $11,000,000 in improving the Chicago River and reducing velocities, and is preparing to do more.

(9) The committee gives the impression that the sewage from certain important areas, such as the Stock Yards, cannot be brought into contact with the flow of the Main Channel until putrefaction is completed, and this is credited to Mr. Wisner. The language used is not specific, because if putrefaction were completed no water would be required for dilution. The committee fails to distinguish between questions of local nuisance due to sludge deposits and questions of dilution and velocity. There is no doubt concerning the point that the minimum dilution fixed by law, 3⅓ cu. ft. per second per 1,000 people, is not sufficient to cover the industrial wastes in addition to the domestic load. The question of ultimate purification of sewage, however, is a different proposition from the question of local nuisance.

The committee did not find out what the real relations were between the Sanitary District and the federal government, and the purposes of the suit referred to in their heading 10. The government has never been sued by the District, because the District does not admit that the government has any rights in the premises. What has happened is that the government has sued to enjoin the District from taking more than a total diversion of 4,167 cu. ft. per second from the lake; in other words, to prevent the District from following its charter requirements. The restriction of the diversion to that figure would render useless a large proportion of the sum spent on the Drainage Canal and its appurtenances, and would in addition require the expenditure of around $100,000,000 in the course of the next ten years to get to the point where the District practically is today. The extra cost of operating would raise the damage to over $200,000,000. This burden of expense seems inequitable and ill considered. The investigations of the District have shown that this expenditure would benefit only a few, namely, the ore shippers through the St. Clair River. Fluctuations in lake levels caused by variations in the rainfall and by winds, and even by the acts of the government engineers themselves, in dredging channels between the lakes, have had a far greater influence on the lake levels than any draft the Sanitary District can ever make.

With wind variations in the lake levels of 1 to 5 ft. in a day, it is inconceivable that a draft of the Sanitary District, which will produce only a theoretical lowering of the lake level would have injured shipping, and as a matter of fact the levels of the lake rose after the opening of the drainage canal for over ten years. The committee is entirely correct in stating that the city of Chicago is facing a grave situation, but there is an apparent immediate practical relief which is well within the control of the citizens of the Sanitary District of Chicago, viz., arousing of sufficient public opinion and interest to wipe out all idea of private gain and force representatives of the people in Congress at Washington to grant definitely to the District a diversion of 14,000 cu. ft. per second. It would seem that at the present day that sanitation and public health were sufficiently entrenched to be put above private gain.

The policy of the Sanitary District of Chicago in extending its boundaries is entirely correct from an engineering standpoint, as the problems being solved by the Sanitary District intimately concern the citizens of Chicago and the citizens of the outlying territory, and as yet no territory has been added which would in the immediate future seriously embarrass the District. The International Waterways Commission said on the best of expert advice, namely, a report signed by Messrs. Hering and Fuller in 1905, that the Calumet region could be cared for both suitably and economically by methods of sewage purification, but both gentlemen were asked to report on methods other than dilution, and both have time and again before and since 1905 reiterated the fact that the best and most economical method for the entire territory of the Sanitary District was sewage diversion and dilution.

During the past two years the Sanitary District has employed a number of the most eminent consulting engineers in the United States, men of recognized standing and repute, of impartial judgment, trained by long experience. Among these has been George W. Fuller, Allen Hazen, F. P. Stearns, Rudolph Hering, Gardner S. Williams and John R. Freeman. The various phases of the water and sewage situation and the lake levels were considered by these experts. The weight of the opinion of these gentlemen is wholly to the fact that at present Chicago has a remarkably good water supply, and that this condition is due entirely to the diversion of the sewage. Sewage disposal by dilution has also been approved after actual inspection of conditions, and the present scheme is warmly endorsed. The expenditure of vast sums of money to replace sewage facilities which would be thrown away by the reduction of the diversion is deprecated, nor is it felt that such a course would put the city back in as good a condition as regards the character of water supply as it is today.

With the aid of this array of engineering advice, supplemented by the engineering crops of the Sanitary District, marked progress has been made along definite lines to keep informed of the conditons existing both here and abroad and learn what can be done, and when the time is ripe to go ahead. So long as the present injunction proceedings are pending, nothing can be done, unless the citizens of the Sanitary District will go over the heads of the courts by awakening the representatives and senators in Congress to pass an act authorizing a diversion of 14,000 cu. ft. per second.

From the standpoint of sewage disposal in Chicago, it is time that the public realized that the history of the art of sewage disposal shows that of all the known processes for this purpose available in 1887 to Chicago and cities of large size, the dilution method was the only one which has stood the acid test of time. City after city in England and on the Continent is abandoning farms. The use of intermittent sand filters has become restricted to small communities, and then only in regions where sand is comparatively cheap, and the process of chemical precipitation is used only under special circumstances, within always narrowing limits. There is perhaps no city in the United States having a sewage plant on which so much depends as is the case in the Sanitary District, nor is there any city of its size or even over 100,000 people in which the works have as yet been operated continuously, as would be required here, if the desires of some were followed. In Baltimore, the works constructed have not received their load of sewage, as the sewer system has only recently been installed, and connections are being made slowly. In Columbus, the works have been operated intermittently, because in the winter the high flows of the river are sufficient to carry along the sewage without creating any objectionable conditions.

Until a diversion by water is agreed upon, either by Congress or by the final decision of the Supreme Court of the United States, only tentative schemes can be discussed, subject to assumed conditions. The authorities of the Health Department of the City of Chicago are thoroughly conversant with the condition of the water, and the authorities of the Sanitary District have followed very carefully the condition of the drainage canal and have consulted the best obtainable advice, and have studied how the results obtained compare with those in other cities. From this mass of information, carefully collected from unbiased sources, there is no doubt that the conditions in Chicago today are not a subject of alarm, and that the scheme followed of diverting the sewage and diluting it with water is the best, most economical, and most efficient means of handling the sewage of the city and protecting the water supply. From the information in the hands of the City and the Sanitary District of Chicago, with the knowledge of the exact diversion that can be had, plans will be readily and properly crystallized, whichever way the matter may develop.

ACTION BY HARBOR AND RIVER IMPROVEMENT COMMITTEE OF THE CHICAGO REAL ESTATE BOARD.

As stated in the opening paragraph of this article, the committee on harbor and river improvement of the Chicago Real Estate Board in its report dated Feb. 4, 1914, advised the employment of three experts not previously interested in Chicago sanitary matters for the purpose of studying and reporting upon a proper sanitary policy for the city, relative to water supply and sewage disposal. A second report by the harbor and river improvement committee dated April 1, 1914, announces the engagement of the three experts. We quote from this report as follows:

> Acting, as we believe, in the best interests of our city, as a whole, and with a view to determining the proper future sanitary policy for this city, not alone as to the questions arising as to its drainage, but those connected with its water supply and public health, we have secured the services of a board of three distinguished sanitary experts, who will convene in this city during the month of April, to study the problems outlined to your Board (the Chicago Real Estate Board) in the reports of the Drainage Committee, presented Jan. 7, 1914, and the further reports of the Harbor and River Committee, presented Feb. 4, 1914.
>
> In view of the recent statement of the President of the Sanitary District that we are facing a possible expenditure of over $200,000,000 for further sanitary protection, in case the U. S. Government suit is adversely decided, we believe that this study is of the utmost importance at the present time.
>
> The personnel of the board (of experts) will be as follows.
>
> Mr. George A. Soper, President of the Metropolitan Sewerage Commission of New York City.
>
> Mr. Arthur J. Martin, Sanitary Expert of London, England, Past President of the British Sanitary Institute.
>
> Mr. James D. Watson of Birmingham, England, President of the Institute of Sanitary Engineers, and long Chief Engineer of the Birmingham, Tame and Rea District Drainage Board.
>
> These gentlemen are all eminent in their profession, and men of long experience in sanitation, and it is believed can bring helpful advice as to the future sanitary protection of this city, not only as to its drainage, but as to its water supply problems.

The findings of Messrs. Soper, Martin and Watson will be reported in this journal. Following the plan here indicated the action and opinions of the engineers for the Sanitary District of Chicago, relative to the forthcoming report by the experts above named, will also be reported in this journal.

Data on Removal of Suspended Matter by Sewage Screens.

Few data have been published which show the efficiency of sewage screening. Most of these data are summarized in the present article. In most cases sewage screens have been installed to prevent injury to or stoppage of pumping machinery. Screens for this service do not require a finer mesh than is formed by bars spaced about one inch apart. In some places it has been found desirable to remove as much of the floating and suspended matter as practicable by means of much finer gratings or screens, since a large proportion of this matter is unstable or slowly putrescible and tends to accumulate in the interstices of sewage filters from which it must be removed at considerable expense. In order to secure information as to the amount of suspended matter removed from sewage by screens of various sized openings Messrs. T. C. Schaetzle and E. R. Davis performed a series of experiments in 1913 upon the sewage of Ithaca, N. Y. The results of these experiments were incorporated in a thesis for a degree from Cornell University. The thesis mentioned also summarized in small space the literature of the subject. An abstract of the thesis mentioned was published in the Cornell Civil Engineer and the information here given is taken from that source.

Engineer E. Kuichling in his report on "The Disposal of the Sewage of the City of Rochester, N. Y.," made in February, 1907, states:

> In general it may be said that it is the suspended matter which imparts to sewage its offensive appearance and causes the serious complaints about the pollution of natural water courses. The quantity of such suspended matter contained in sewage varies widely in different places, and also at different seasons, days and hours in the same place.

The Cornell experiments conclusively demonstrate the truth of the foregoing statement as regards hourly variation. This variation in quantity and quality during 24 hours was plainly shown by Messrs. Hazen and Whipple in their experiments on the sewage of Paterson, N. J., made in 1906. They showed that at a number of different points in the city the dried suspended matter ranged from 15 to 641 p. p. m. by weight. Similarly, in the report on the purification of the sewage at Columbus, Ohio, made in 1905 by George A. Johnson, it is shown from daily analysis extending over a period of ten months that the dried suspended matter ranged from 12 to 842 p. p. m., with an average of 209 parts.

Although the quantity of suspended matter that can be removed from sewage by screening has usually been regarded as insignificant, owing to the relatively large width of the spaces between the bars of the gratings ordinarily used, during the last decade the use of fine screens has made great headway in Germany. In a discussion by Kenneth Allen, engineer of the Metropolitan Sewerage Commission of New York City, he says:

> At present this may almost be spoken of as the standard treatment in that country. Elsewhere little has been done in this direction, although the Weand screen was adopted at Reading, Pa. Of the various types of German screens the rotary screen employed at Frankfort and the inclined, perforated plate used at Dresden are the most familiar to American engineers.
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> Less familiar in America is the drum screen invented by G. Windschild of Dresden. In this screen some of the objections to fine screens appear to have been overcome. They are, therefore, more nearly in competition with settling tanks for the removal of suspended solids.

Mr. Allen believes that there is a definite, though somewhat limited, field for the employment of fine screening in America. The removal of even 20 to 25 per cent of the suspended matter will render many sewages fit for discharge into bodies of water affording ample dilution.

ITHACA RESULTS.

At Ithaca all sewage passes to a pumping station and is pumped into septic tanks. As the sewage enters the pumping station it passes through a bar screen with ¾-in. openings. This screen removes rags, sticks, fruit peelings, etc. Between the pumps and septic tanks a ½-in. mesh screen was installed. The screenings removed from this screen fluctuated in amount from hour to hour. Thus at 4 a. m. and 11 a. m. the amount of screenings removed was 2.6 lbs. and 103.97 lbs. per 1,000,000 gals. respectively. These figures are for dry screenings. The moisture content of the screenings ranged from 42.8 per cent to 89.63 per cent, with an average of 79.4 per cent for all runs taken. In studying the ratio of flow to suspended matter removed it was observed that in general with an increase in flow there is an increase in screenings removed. Likewise there is a general fluctuation in the percentage of moisture as the amount of screenings removed increases or decreases, though remaining close to 80 per cent.

COMPILATION OF SCREENING DATA FROM VARIOUS CITIES.

At Providence, R. I., the matter collected on a grating, the bars of which are 1 in. apart, was at the rate of 41 lbs. per 1,000,000 gals. in 1906; at the Dorchester sewage pumping station in Boston the rate in 1897 was 300 lbs. per 1,000,000 gals., with a similar spacing, whereas at the several pumping stations of the Metropolitan Sewerage System of Boston, with screens composed of ¾-in. bars, 1½ ins. center to center, and a similar series staggered in front of these, providing an effective clear space of ⅛ in., the results obtained varied from 135 to 397 lbs. removed per 1,000,000 gals. of sewage. At the northern outfall works of the London sewers it was 213 lbs. with a grating consisting of bars spaced ¾ in. apart; at the pumping stations of the Berlin sewerage system the rate in 1900 was 212 lbs. with bars spaced 0.6 in. apart; and at Charlottenburg it was about 200 lbs. in 1902.

As stated previously, the clogging of filters has recently directed the attention to the development of a satisfactory system of fine screening whereby sewage treatment may be facilitated and rendered less expensive. Along this line experiments have been made with plates as well as stationary and revolving screens.

In 1898 experiments were made at Leeds, England, with zinc plates perforated with ⅛-in. holes, resulting in obtaining 1.33 cu. yds. of well-drained matter per 1,000,000 gals., or at the rate of 30 lbs. per cubic foot it removed 1,080 lbs. per 1,000,000 gals. Similar results were obtained at Weisbaden, Germany, during 1902 by passing the sewage of the entire city through a set of three screens having clear openings of 0.60, 0.16 and 0.04 in. respectively. The material collected amounted to 1.35 cu. yds. or 1,098 lbs. per 1,000,000 gals. In 1904 these screens were remodeled and changed to four in number with clear openings of 1.6, 0.60, 0.10 and 0.06 in. From these 1.95 cu. yds. or 1,581 lbs. per 1,000,000 gals. were obtained. Experiments were also made at Dresden in 1906 with a large revolving screen of sheet metal perforated with slots 0.08 in. wide, and it was found to collect an average of 0.955 cu. yds. or 692 lbs. per 1,000,000 gals. during continuous service. The screen was a circular disc 15.1 ft. in diameter, inclined 15° from the horizontal so that about one half its surface was submerged. In all the above cases the moisture content was about 75 per cent.

With a similar device the collection at Coventry, England, was reported to be from 70 to 300 lbs. per 1,000,000 gals. In 1899 a screen of an endless belt of wire cloth with meshes about 0.5 in. square was used at Sutton, England, with the result of removing 1,200 lbs. per 1,000,000 gals., and in 1901 a similar apparatus with meshes 0.1 in. square extracted matter at Gottingen, Germany, at the rate of 0.545 cu. yds. or 336.6 lbs. per 1,000,000 gals.

An interesting experiment was made in 1898 by [illegible] Metzger in a German town of 50,000 inhabitants called Bromberg, with a wire cloth screen having meshes only 0.06 in. sq. The whole of the sewage was passed through the apparatus, both when the screen was kept stationary and when it moved like an endless belt in the outlet sewer. When stationary the maximum collection in one hour was 13,770 lbs. per 1,000,000 gals.; during four consecutive hours 9,511 lbs. per 1,000,000 gals., and the minimum rate in one hour 207 lbs. per 1,000,000 gals. When the screen moved through the sewage, the velocities ranging from 0.5 to 2.0 ins. per second, the collection was 14 per cent less during the four hours of greatest flow in dry weather.

In this country experiments have been made at several places. At Reading, Pa., the crude sewage is mostly domestic in character, although it contains a little trade wastes but no street washings. The screen is a revolving horizontal cylindrical frame covered with brass wire cloth having meshes about 1/40 in. square. The quantity of solid matter collected in 1908 was reported as being 0.698 cu. yds. per 1,000,000 gals.

During the investigations of the sewage at Columbus, Ohio, in 1904-1905 the liquid examined was passed through a screen having meshes 0.5 in. sq., the same being placed in the main outlet sewer parallel with the line of flow to facilitate its cleaning. The report states that this screen never became clogged sufficiently to require attention. Considerable coarse material which passed through this screen was caught by the screens in the screen chamber, necessitating the cleaning of these screens from five to six times each day. About 300 lbs. or 0.17 cu. yd. of wet screenings were thus removed on an average per 1,000,000 gals. of sewage. In the report these screens are described as being two in number, set vertically near the outlet end of the chamber, and formed of "diamond mesh wire cloth woven with No. 12 wire. The first screen had a clear opening of 0.5 in. and the second 0.375 in." From these data the wet screenings amounted to 0.17 cu. yd. per 1,000,000 gals.

At the Philadelphia experimental plant the sewage was delivered through 21 ¼-in. nozzles upon the conical surface of a screen having 32 meshes per inch (clear openings 0.5 mm. square). From September, 1909, to April, 1910, the screen removed the equivalent of 500 lbs. of dry solids per 1,000,000 gals. of sewage.

The effect of screening on subsequent treatment was found to be (according to Fulsier, Spillner and Allen):

1. A more uniform sewage by eliminating in part the irregularities due to trade wastes.
2. A reduction in the sludge subsequently treated.
3. An increase of the moisture in the sludge subsequently treated.
4. A finer subsequent and one more readily pumped.
5. An entire absence during nine months of clogging sprinkler nozzles using settled screened sewage.

At Waterbury with a ½-in. [illegible] wire nothing of particular importance was noted except that 110 lbs. [illegible] cu. yds. of screenings were removed [illegible] 1,000,000 gals. of sewage. At Pl[illegible] with openings in the screen of ½ in. the [illegible] with a moisture content of 85 [illegible] was removed per 1,000,000 gals.

At Pawtucket, R. I., the matter removed by a screen composed of wooden strips ¾ in. [illegible] spaced ⅜ in. apart ranged from 40 to 1,071 lbs. of dry solids per 1,000,000 gals. of sewage.

In the experiments at Ithaca from March to May, 1913, the material removed per 1,000,000 gals. ranged wet, from [illegible] per 1,000,000 gals. with an average [illegible]. When dry the [illegible] varied [illegible] 188 lbs. with an average of 57 lbs. per 1,000,000 gals.

In conclusion it might be said that with bar screens having a clear space from ⅜ in. [illegible] the weight of wet screenings removed is likely to range from 11 to 367 lbs. per 1,000,000 gals.

With plates and wire screens the weight removed wet ranges from 110 to 13,770 lbs. per 1,000,000 gals. sewage, whereas the volumes vary between the limits of 0.08 to 10.25 cu. yds. wet screenings per 1,000,000 gals. When considered as the number of parts per million removed the limits ranged from 9.0 and 305.2, while in most cases the screenings contained 75 per cent moisture. Engineer E. Kuichling says:

> In general it has been found that with the usual coarse gratings at sewage pumping stations, the quantity of suspended matter arrested is from 3 to 10 cu. ft. per 1,000,000 gals., and that such matter in its wet state ranges in weight from 20 to 65 lbs. per cubic foot. On being completely dried, it loses from 50 to 75 per cent of its weight, so that on an average it can be assumed that such coarse screening removes about 104 lbs. of dried matter per 1,000,000 gals., or about 12.5 parts by weight per million. Most of this matter is organic and more or less quickly putrescible in character.

Fine screens, however, remove a much larger amount proportionately, but owing to the difficulty in keeping the finer mesh screens clean the coarser materials, such as towels, sticks, peelings, etc., are first removed by bar screens so that the proportion removed by the finer screens as they are now operated does not give a fair test of just what quantity of suspended matter they will remove.

GENERAL CONCLUSIONS.

Mr. Geo. W. Fuller states that for sewage of about 90 gals. per capita per day screens with clear spaces of 0.5 in. will remove from 5 to 6 cu. ft. of screenings per 1,000,000 gals. of sewage. Likewise, that in order to cause an effective removal of organic matter the clear openings must be less than 0.4 in.

Mr. E. Kuichling cites the case of Hamburg as an illustration of what can be accomplished by moderately fine screens. He says:

> Hamburg is a municipality with a present population of over 820,000 and an area of about 30 square miles. It is located on the River Elbe, and its sewage has for many years been discharged in the crude state into the river whose minimum flow is about 5,200 cu. ft. per second. It is also one of the busiest ports for shipping in the world, and its docks and wharves are always crowded with vessels. To relieve the tidal waters of some of the pollution, the sewage has for several years past been screened through gratings with bars spaced about 0.6 in. apart, and the effluent is discharged at the bottom of the river through three submerged steel pipes 6.56 ft. in diameter, extending respectively 2,300; 328 and 436 ft. out from the shore line. The result has been entirely satisfactory and it is now reported to be almost impossible to detect any evidence of sewage pollution in traversing the river with a vessel. The suspended matters in the sewage that pass through the screen seem to remain near the bottom, and to be finally carried out to sea by the current.

The screenings from screening plants differ greatly both in amount and in character. The meshes of the screen or the spacing of the bars may run from 1.0 in. (25mm.) to 0.04 in. (1 mm.), according to whether it is desired to keep coarse matter from the pump, or to secure the greatest possible clarification of the sewage to prevent the clogging of filters.

The amount varies with the days of the week, as well as during the course of the day. Since most of the coarser suspended matter consists of wastes from habitations, the amount reaches its maximum about noon, or shortly before, and commences to disappear about midnight, reaching its minimum between 4 and 5 o'clock in the morning. The material is almost wholly organic and consists of scraps of vegetables or fruit, cloth, hair, wood, paper and lumps of fecal matter.

The amount of material varies so widely that it is impossible to state just what to expect any definite sized screen to remove.

The amount of water contained is small as compared with that in the sludge of sedimentation basins and amounts to from 70 to 80 per cent. On account of its organic origin it is highly putrescible.

The weight of the screenings removed depends to a large extent upon the thoroughness with which they are drained before being weighed.

Railways

Principles and Methods for the Valuation of Railroad Property and Other Public Utilities.

(Continued from Page 420.)

APPRECIATION.

The valuation of some items of property, such as land and water rights, differs from the valuation of other property in several respects. Land in most situations increases in value from year to year, especially in growing cities. Terminal land of railroads and land of other public service corporations sometimes increase in value so rapidly that the question arises as to the justice of basing rates upon the present value of the land rather than its original cost.

Your committee is aware that the decisions of the higher courts indicate that present prices should be used in making a valuation of land as well as of structural property, and that the present practice generally gives to the corporation as to the individual the so-called unearned increment due to appreciation in land values

This in incompatible with the fundamental principle that the corporation is entitled to a fair return upon the fair value of its property, because, if the land is valued at the increased price, and the appreciation in the value of the land is not included in the accounting, the corporation would receive a sum in addition to a fair return amounting to the appreciation in the value of the land.

The courts have recognized that the corporation is entitled to earn from the public the sum necessary to offset the depreciation in the value of its structural property; similarly, the public is entitled to receive from the corporation due recognition of the increase in value of those portions of its property which appreciate in value. The principle involved in the two cases is the same, and there is no reason to think that its application to both would not be supported by the higher courts.

Your committee therefore recommends as the best way of conforming to the equities of the case under commission control, that the land or other property which appreciates in value be valued at present prices, and that the appreciation in the value of such property be offset against the depreciation in the value of the other property. This method would keep the valuation of the land or other appreciating property approximately commensurate with its market value from time to time, and the corporation, having in effect paid to the public the amount of the appreciation, would thereafter be entitled to earn a fair return on the appreciated value of the property in exactly the same way that, having been paid by the public the amount of the annual depreciation of the other property, the corporation must accept a return upon the depreciated value of such property.

Attention is called to a very clear discussion of this question by Commissioner Maltbie of the New York Public Service Commission for the First District, *in re* Gas and Electric Rates of the Queens Borough Gas and Electric Co.[*]

> Land differs from most property in that it generally appreciates in value, and the question has been raised, whether land should be included in "fair value" in rate cases at its original cost or at its estimated value at the time the rate is to be fixed. It is well settled that other property should be taken at its *then* value, but it has been argued that in the case of land the original cost should be used. While it is evident, therefore, that each case must be decided upon the facts peculiar to it, the Commission believes it proper in this case to follow the general rule, as stated by Judge Hough of the United States Circuit Court (Consolidated Gas Co. v. City of New York, 157, Fed. Rep., 855):
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> "Upon reason, it seems clear that in solving this equation the plus and minus quantities should be equally considered, and appreciation and depreciation treated alike. Nor can I conceive of a case to which this procedure is more appropriate than the one at bar."
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> Thus, land has been taken at its fair value and not at its original cost, and the annual appreciation of land has been treated as a profit. By this method, all property is treated absolutely alike, as Judge Hough suggests. No difference is made, except that as depreciation represents a decrease in assets, it is placed as a *debit* against operation, while appreciation is placed as *credit* because it is an increase in assets. Land has sometimes been treated like other property only to a degree; that is, each class has been appraised at its present worth or value. That has been done in this case. But if property is to be taken at its *depreciated* value where it has depreciated, an entry must regularly be made in estimated operating expenses equal to the average annual depreciation. Conversely, if land, or any other property which genuinely appreciates in value, is to be taken at its *appreciated* value, then an entry must be made in the estimated receipts equal to the average annual appreciation. Unless this is done, it is obvious that the consumer will be burdened with all the estimated decreases in assets but not credited with the increases in assets. If the principle laid down by the courts is to be followed in part, it should be followed in whole.
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> It is suggested that the annual increase in the value of land which is treated as income is not actually received. Increase in the value of unoccupied land is not realized until sold or put into use, but it is real, nevertheless, although payment may be deferred. Likewise, payments to the depreciation fund are not actually expended; yet they have been considered legitimate charges in practically every case. Furthermore, the *annual* increment is no more indefinite than the *total* increment—the present value. But if the present value can be determined, it is possible to determine past *annual* appreciation with positive accuracy, for it is only a simple mathematical calculation. It is also probably as easy to estimate increase in the near future as it is to estimate what obsolescence, which is a form of depreciation, there will be in the future.
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> Indeed, the problem of handling appreciation is much simpler than depreciation. If the property is growing more valuable, the investor need not worry; and if the state recognizes his right to earn a fair return upon the increase, he is fully protected. It is not necessary that the increase be represented by stocks or bonds, for if the earning power is there, he will receive a return thereon, regardless of the amount of securities. In fact, the existence of an increase which is not represented by securities is an element of safety, a reserve fund of a valuable kind.
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> There is a further similarity. The exact amount of depreciation and the annual rate are not definitely known until the piece of property is actually replaced or has become useless. The total appreciation and the average annual rate are not known until the land is sold, but when it has been disposed of (and plants are contin-

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[*] P. S. C., 1st D., June 23, 1911.

ally being removed and the land sold) they become absolute certainties. Why should these matters be considered less definite when applied to land than when applied to the buildings thereon? The depreciation of the buildings is a charge against operation; why should not the appreciation of land be a credit?

METHOD OF VALUING LAND.

It is more difficult to appraise correctly public service land than to appraise the structural property, as the views of different appraisers of land vary widely. For this reason and because of the decisions of the courts on these matters, to be discussed subsequently, it is the view of the committee that in cases where land has been purchased in fairly recent times and its cost is known, that the present value of the land should be based upon its actual cost plus its subsequent appreciation in value, such appreciation to correspond to that of neighboring property.

Where the valuation cannot be made in this way, it is the view of the committee that land can be valued most correctly on the basis of present prices of neighboring land of similar character, augmented by the ratio ordinarily found to obtain in that region between land acquired by public service corporations, on the one hand, and by private parties on the other, and subject to the value of improvements upon the land obtaining at the time of purchase, said improvements being appraised at their present day prices.

Due consideration also should be given to the increased cost and value of public service property where it has been necessary to take a strip of land running through a large tract. In such a case the severance of the property into two parts decreases the value of the land not taken and, therefore, augments the cost and value of the strip of land acquired.

In applying either of the above methods, care should be taken to use, as far as practicable, as a basis for valuation, land which has not been affected in value by the use to which the public service property is devoted.

Decisions of Courts and Commissions Relating to Land.—In the valuation of land, courts and commissions have generally adopted present values instead of original cost.

The decision of the United States Supreme Court in the Consolidated Gas Co. case[*] contains this statement:

> And we concur with the court below in holding that the value of the property is to be determined as of the time when the inquiry is made regarding the rates. If the property which legally enters into the consideration of the question of rates has increased in value since it was acquired, the company is entitled to the benefit of such increase. This is, at any rate, the general rule. We do not say there may not possibly be an exception to it where the property may have increased so enormously in value as to render a rate permitting a reasonable return upon such increased value unjust to the public. How such facts should be treated is not a question now before us, as this case does not present it.

In the case referred to, real estate was an important item, as it represented upward of 40 per cent of the value of the tangible property.

On June 9, 1913, the opinion[†] of the United States Supreme Court in the Minnesota Rate cases was delivered by Mr. Justice Hughes. This opinion, in its treatment of land values of a railroad corporation, is so far reaching and important that it will be discussed at length.

Three similar cases are included in the opinion. The suits were brought by stockholders of the Northern Pacific Railway Co. and of two other companies, respectively, to restrain the enforcement of orders of the Railroad and Warehouse Commission of the State of Minnesota prescribing maximum charges for the transportation of freight and passengers. The cases had been heard before a master and decided by the United States Circuit Court in favor of the railroads. They were appealed to the Supreme Court and the decision of the Circuit Court reversed in two instances. In the third instance it was sustained on the ground that the prescribed rates were confiscatory. The case of the stockholders against the Northern Pacific Railway Co. was used as the basis for the extended discussion of land values.

The lower court found that the value of the portion of the railway system in the State of Minnesota was, in round numbers, $90,000,000, and in making up this valuation, land for right of way, yards and terminals was included at $21,000,000, exclusive of overhead charges.

The net profits of the company for the year ending June 30, 1908, were equal to 6 per cent on the entire estimated value of the property.

A very large part of the land valued was terminal property in St. Paul, Minneapolis, and Duluth, the total for these terminals amounting to $17,300,000, out of the total of $21,000,000. The original cost of the terminal property was $4,500,000.

The testimony upon which the valuation of the land was based was to the effect that experience had shown that where a railroad company is compelled to obtain a given property it has to pay much more than the market value of contiguous or similarly situated property, and a witness used a multiple to show the relation between these values. In addition, where a strip or other portion of a property is required, the railway company has necessarily to pay for damages to the remaining property, caused by severance or otherwise, and a further multiple was used to cover this feature.

The court was dealing with property which as a whole had increased greatly in value since its acquisition and probably with widely conflicting estimates of value[*]. Terminal lands in large cities represented by far the greater part of the total value of the property. This statement is made in the opinion in regard to the Cost-of-Reproduction Method of valuing this property (page 452):

> Moreover, it is manifest that an attempt to estimate what would be the actual cost of acquiring the right of way, if the railroad were not there, is to indulge in mere speculation. The railroad has long been established, to it have been linked the activities of agriculture, industry and trade. Communities have long been dependent upon its service, and their growth and development have been conditioned upon the facilities it has provided. The uses of property in the communities which it serves are to a large degree determined by it. The values of property along its line largely depend upon its existence. It is an integral part of the communal life. The assumption of its non-existence, and at the same time that the values that rest upon it remain unchanged, is impossible and can not be entertained. The conditions of ownership of the property and the amounts which would have to be paid in acquiring the right of way, supposing the railroad to be removed, are wholly beyond reach of any process of rational determination. The cost of reproduction method is of service in ascertaining the present value of the plant, when it is reasonably applied and when the cost of reproducing the property may be ascertained with a proper degree of certainty. But it does not justify the acceptance of results which depend upon mere conjecture. It is fundamental that the judicial power to declare legislative action invalid upon constitutional grounds is to be exercised only in clear cases. The constitutional invalidity must be manifest, and if it rests upon disputed questions of fact the invalidating facts must be proved.

This opinion, when taken literally, appears to be rather drastic, but taken in connection with the cases under discussion it is not unreasonable.

Some of the objections to the reproduction method here set forth apparently would not apply to the case of aqueduct or reservoir properties, which have little or no effect upon the value of lands near them, except as such value is decreased by severance. It should not affect the valuation of strips of land for railroads along those portions of the line where the value of the property has not been materially affected by reason of the presence of the railroad. Other objections would not apply if rational and equitable theories were used by the experts making the valuation.

Two other related portions of the opinion are (pages 450 and 453):

> It is at once apparent that, so far as the estimate rests upon a supposed compulsory feature of the acquisition, it cannot be sustained. It is said that the company would be compelled to pay more than what is the normal market value of property in transactions between private parties; that it would lack the freedom they enjoy, and, in view of its needs, it would have to give a higher price. It is also said that this price would be in excess of the present market value of contiguous or similarly situated property. It might well be asked, who shall describe the conditions that would exist, or the exigencies of the hypothetical owners of the property, on the assumption that the railroad were removed? But, aside from this, it is impossible to assume, in making a judicial finding of what it would cost to acquire the property, that the company would be compelled to pay more than its fair market value. It is equipped with the governmental power of eminent domain. In view of its public purpose, it has been granted this privilege in order to prevent advantage being taken of its necessities. It would be free to stand upon its legal rights and it cannot be supposed that they would be disregarded.
>
> That question [the substantial question to be decided] is whether, in determining the fair present value of the property of the railroad company as a basis of its charges to the public, it is entitled to a valuation of its right-of-way not only in excess of the amount invested in it, but also in excess of the market value of contiguous and similarly situated property. For the purpose of making rates, is its land devoted to the public use to be treated [irrespective of improvements] not only as increasing in value by reason of the activities and general prosperity of the community, but as constantly outstripping in this increase all neighboring lands of like character, devoted to other uses?

The rule laid down in the above quotations, that nothing can be allowed in valuing public service property for the additional cost on account of the compulsory feature of the acquisition of the property, is one which, in some cases, would do great injustice to the corporation if the original cost of property were not known and it were necessary to ascertain its value by other means.

It is a matter of common knowledge to engineers and others connected with the acquisition of right of way or of tracts of land with specified boundaries, that the municipality or public service corporation, even if exercising the right of eminent domain, is compelled to pay much more for such property than the [illegible] market value of the land acquired.

[illegible] take [illegible] instance, namely, the ac[illegible] of real estate for the Wachusett Reservoir in Massachusetts.

In order to make a preliminary estimate of the cost of acquiring the real estate, the first step, after the preparation of plans showing the properties, was the selection of one man as an appraiser in each of the municipalities in which the site of the reservoir was situated, choosing in each case the man best acquainted with the properties and their value. These experts, some of whom owned property required for the reservoir, were instructed to value all of the property, except mills and water rights, on a liberal basis. The aggregate valuation so obtained was:

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[*] Wilcox v. Consolidated Gas Co., 212 U. S., 19, January 1, 1909.

[†] Simpson et al. v. Shepard; same v. Kennedy; same v. Shillaber, 230 U. S. 352, June 9, 1913. This opinion has also been published as Senate Document No. 54, 63d Cong., First Sess.

[*] In a paper by Mr. C. F. Staples, member of Minnesota Railroad and Warehouse Commission, dated September 15, 1913, and entitled, "The Unreliability of Unsupported Opinion Evidence in Determining Land Values in Railroad Appraisals," comparisons are made between the expert opinion evidence, in behalf of respondents and petitioner, of the value of a given terminal property in the city of St. Paul. Seven witnesses for the respondent valued the property at from $7[illegible],341 to $287,013, with an average of $159,385, while seven other witnesses for the petitioner placed the value at from $[illegible],003 to $6,395, with an average of $3,571. The jury in this case made an award of $6[illegible],916.

The paper also refers to high-class terminal lands in St. Paul and Minneapolis bought at a comparatively recent date (apparently between 190[illegible] and 1906) which cost $3,06[illegible],239, and these lands were valued as of June 30, 1906, by six real estate experts employed by the railroads at $10,101,831, equal to 329.2 per cent of the actual cost to the carriers, although it is stated that "there was no material advance in value from 1900 to 1906, inclusive."

Land (4.772 acres)	$244,000	
Buildings	453,000	$697,000

The cost of acquiring the same property, exclusive of overhead charges, was $1,182,000, which was $485,000, equal to 69 per cent, in excess of the original liberal appraisal of the value. This excess, which was due to the compulsory feature of the acquisition, was not wholly unexpected, as in the preliminary estimates of the cost of the property 50 per cent was added to the experts' valuation, and to the sum so obtained 15 per cent was added for overhead charges, exclusive of interest, making a total of $1,200,000.

If it is necessary, in making a preliminary estimate, to add a percentage to the ordinary value of the property to cover the extra cost when the taking is compulsory, why should not a similar percentage be added to the ordinary value of the property in estimating the cost of reproduction?

If the real estate had all been land, and the experts' valuation had been based on actual sales rather than a liberal market price, it is probable that the cost of acquisition would have been from two to three times the preliminary valuation.

This reservoir is situated in a part of a state where land values are very nearly stationary, and any attempt to appraise at the present time on the basis of the value of contiguous and similarly situated property, without the use of multipliers, would result in a valuation much less than the original cost of the property.

The situation is so very different from that involved in the valuation of terminal properties in the Minnesota Rate Cases, that it hardly seems possible that with the facts presented the courts could do the injustice which would result from neglecting the compulsory feature as affecting the cost of acquiring such property.

The hasty reader of the opinion under consideration might easily reach the conclusion that under no circumstances would a value of public service property in excess of the market value of contiguous and similarly situated property be approved. The portions of the opinion referred to, given without the context, are as follows:

> It is also said that the price [a price necessarily paid for property] would be in excess of the present market value of contiguous or similarly situated property. It might well be asked, who shall describe the conditions that would exist, or the exigencies of the hypothetical owners of the property, on the assumption that the railroad were removed?
>
> That question is whether * * * the railroad company * * * is entitled to a valuation of its right-of-way not only in excess of the amount invested in it, but also in excess of the market value of contiguous and similarly situated property.

This question is answered in part by the following statement:

> It is not admissible to attribute to the property owned by the carriers a speculative increment of value over the amount invested in it and beyond the value of similar property owned by others, solely by reason of the fact that it is used in the public service.
>
> The increase [in value] so allowed * * * cannot properly extend beyond the fair average of the normal market value of land in the vicinity having a similar character.
>
> The company would certainly have no ground of complaint if it were allowed a value for these lands equal to the fair average market value of similar land in the vicinity, without additions by the use of multipliers or otherwise to cover hypothetical outlays.

The language employed is somewhat perplexing, but when the above extracts from the opinion are read in connection with the context and are also considered in connection with the circumstances of the case, it seems clear that they are not to be construed as meaning that the courts would not permit valuations, for instance, of strips of land, at a price in excess of the value of contiguous and similarly situated property.

When a strip of land is taken from the middle of a large tract, the owner of the tract is entitled, legally and equitably, not only to the value of the strip based upon the price of similar and contiguous land, but he is entitled to the consequential damages to the remainder of the tract, all of which enhances the cost and consequently the value of the strip taken. It is obvious that the court could not under such circumstances refuse to recognize such extra cost and value of the strip.

Still another part of the opinion calls attention to a further increase in the valuation of land which had been approved by the lower court. The statement is made that the amount allowed for real estate, exclusive of overhead charges (page 450):

> Was included in the total on which 4½ per cent was allowed in Item 30 for "Engineering, superintendence, legal expenses," and again was included in the total on which 5 per cent was allowed in Item 37 for "Contingencies," and, in addition, was included in the total on which 10 per cent was allowed in Item 39 for "Interest during construction."

Elsewhere the opinion states (page 455):

> We also think it was error to add to the amount taken as the present value of the lands the further sums calculated on that value, which were embraced in the items of "engineering, superintendence, legal expenses," "contingencies," and "interest during construction."

If this ruling, instead of being applicable to the particular circumstances of the case under consideration, is intended to have a general application, it is, in the judgment of the committee, inequitable because the items of cost "engineering, superintendence, legal expenses," "contingencies" and "interest during construction" are real items of cost in the acquisition of land, and should be included in any attempt to determine the value of the land on the basis of the cost of reproducing the property, if correct results are to be attained.

The committee believes that the proper purpose of a cost of reproduction estimate of the value of real estate is to ascertain the cost of acquiring such real estate under the conditions existing at the time of its acquisition, except that if it be admitted that the property is to share in the increase in the value of neighboring land not affected by the use to which the public service property is devoted, it is necessary to use the present rather than the original prices. In order to do this the overhead charges must necessarily be included.

The detailed records of the cost of acquisition of a large tract of real estate are available in the case of the Wachusett Reservoir, as shown by the following statement covering all payments from 1895 to Dec. 1, 1909:

Direct Payments for Property Acquired.

Mill property and water rights, including all land and buildings owned in connection with the mills			$1,402,000
Paid for real estate in reservoir and margins			936,200
Paid for real estate beyond the margins of the reservoir			503,700
Total			$2,841,900

Overhead Charges

Administration expenses		1.76%	$50,000
Engineering		2.94%	83,600
Legal and expert		4.67%	132,800
Taxes and tax equivalents		2.24%	63,600
Care of property during construction	$84,700		
Less credits for rents, etc.	67,600	0.60%	17,100
		12.21%	$3,189,000

To the above overhead charges there should be added the portion of the expenses of the State Treasurer's office and of the State Legal Department, attributable to this work which were paid for out of general funds and therefore did not appear in these accounts. This would increase the above overhead charges to at least 13 per cent, and after deducting 2.24 per cent attributable to taxes and tax equivalents, there would remain 10.76 per cent for "engineering, superintendence, legal expenses."

Omissions and contingencies which are necessary items in an estimate are, in the case of book accounts, included in the actual payments and do not appear separately. Interest during construction does not appear in these accounts, but they furnish the data from which it may be computed.

The reservoir, although not completed, was sufficiently far advanced at the end of 1903 so that it could be utilized to a limited extent, and this date has been fixed as the date of operation.

Payments on account of real estate in the preceding years were as follows:

1896	$903,000
1897	642,000
1898	482,000
1899	375,000
1900	229,000
1901	147,000
1902	170,000
1903	134,000
	$3,082,000

To conform with the rate of interest generally used in this report, it will be assumed that the interest on these payments was 5 per cent compounded yearly, instead of the smaller amounts actually paid. On this basis the interest payments during construction would have been $957,000, making the charge for interest during construction 31 per cent of the total of all expenditures to the time when the property was first operated. This is, of course, an unusual charge for interest during construction. Much of the property was acquired a long time before it was needed because more advantageous settlements could be made by so doing.

The real estate in the neighborhood of this reservoir, as already stated, has changed little in value since the time of its acquisition, and if it were to be valued at the present time on the basis of the cost of reproduction without including the extra cost due to the compulsory feature of the acquisition and to the overhead charges, the resulting value would be approximately half its actual cost. It hardly seems possible, in a case like that just stated, that if the cost were not known and it were necessary to determine the value of the real estate on the basis of that of surrounding property, the court would sustain a ruling, which, when literally applied, would confiscate approximately half the value of the real estate.

The opinion itself indicates that the court would accept the full amount of the actual investment (which would necessarily include the compulsory feature and the overhead charges) unless such investment was reckless, or improvident, or improper in some other way. It also clearly recognizes that the present value is not limited to the amount of the actual investment, but involves the recognition of the fair value of the property if it be more than its cost. These features are shown by the following quotations (pages 454, 455)

> It is clear that in ascertaining the present value we are not limited to the consideration of the amount of the actual investment. If that has been reckless or improvident, losses may be sustained which the community does not underwrite. As the company may not be protected in its actual investment, if the value of its property be plainly less, so the making of a just return for the use of the property involves the recognition of its fair value if it be more than its cost. The property is held in private ownership and it is that property, and not the original cost of it, of which the owner may not be deprived, without due process of law
>
> Assuming that the company is entitled to a reasonable share in the general prosperity of the communities which it serves, and thus to attribute to its property an increase in value, still the increase so allowed, apart from any improvements it may make, cannot properly extend beyond the fair average of the normal market of land in the vicinity having a similar character. Otherwise we enter the realm of mere conjecture.

Until the opinion given in the Minnesota

Rate Cases shall have been materially modified by the Supreme Court, it is especially desirable, in the case of all properties of fairly recent origin, to ascertain the actual cost of acquiring the real estate, and to use such actual cost as a basis, adding to the cost of the land an increase in value proportionate to that of similarly situated land not affected in value by the use to which the public service properly is devoted.

Special Value of Land.—It is sometimes claimed that land has a special value for railroad, reservoir, or other public service purposes, either by reason of its special availability and adaptability for the purpose for which it is to be used, or because it is an inter-connected body of land specially available and adapted by shape and location for the special use.

The committee believes that this special value should not be included in a valuation, except to the extent of recognizing all the elements which cause such property to cost much more than ordinary property, but if the actual cost has been augmented by a recognition of the special value of land for the purpose for which it is acquired, allowance should be made for such added cost and value. The case is one where the conditions which prevailed at the time of the acquisition of the property should control rather than the present conditions.

Court Decisions as to Special Value of Land.—A leading case is that of Boom Co. v. Patterson, decided years ago by the United States Supreme Court. It was held that Patterson, who owned three islands in the Mississippi River above St. Anthony Falls, about an eighth of a mile from its western bank, was entitled to payment, not only for the value of the islands as such, about $300, but for their availability and adaptability, when connected with one another and with the main land by booms, for creating a receptacle in which millions of logs could be stored, and the court affirmed a judgment which added about $5,500 for the availability and adaptability of these islands for the purpose stated.

In the arid lands, where irrigation is necessary for the growth of crops, there are many properties containing small reservoir sites and in the Eastern states there are properties along streams where there are utilized or unutilized falls. Such properties are bought and sold and have a market value, and are properly valued at a special price on account of their availability and adaptability for these purposes.

It is now claimed in some cases that land suitable for a site for a great dam, or land forming a part of a great reservoir site, or again, land available for a railroad location through a narrow pass in the mountains, has a special value by reason of its availability and adaptability for these purposes. This claim is urged even where the dam and reservoir are of such magnitude and involve so great a cost that they could not be built except by the community acquiring the property, and where the pass through the mountains is so located that the possibility is remote that it would ever be used by a railroad.

In nearly all cases the availability and adaptability of these properties for the purposes named are not recognized until preliminary investigations are made and action is taken thereon which finally results in their selection.

Up to the time of such investigations the market value of the property is independent of any consideration of such availability and adaptability. Between the time of such investigation and the actual taking of the property by condemnation, there is sometimes an interval of years during which some of the property may be sold to speculators who have knowledge of the enhanced price usually paid for property when the taking is compulsory, and thereby what is in a sense a new market value may be created.

Occurrences during this interval of time may furnish a basis for testimony that there has been a recognition of the special value of the property for the purposes for which it is taken, but if so, it is an added value incidental to the necessary interval of time between the investigation and taking based upon the necessities of the buyer, and is one which should not be recognized. In order to receive consideration, the special value should have existed before the beginning of the earliest proceedings which led to the taking.

Assuming, for the purpose of discussion, that the land taken has a special value to the community or public utility requiring it, because better adapted for the proposed uses than any other, and that this special availability and adaptability should be considered in determining the value of the property, how is it possible to determine the amount of such additional value by other than the most speculative and hypothetical methods? One method sometimes used is that of comparing the cost of works at the selected site with the cost of works at a site which is less available and adapted to the purpose. For example, it may be assumed that the place selected for the site of a dam is in the narrow part of a gorge where 20 acres required for the construction of the dam would, for ordinary purposes, be valued at $100 an acre, or $2,000 for the whole property. Assuming, further, that the most available of other sites is at a point up stream or down stream where the cost of the dam would be increased by $1,000,000, how could the sum so ascertained be used to determine the special value of the better site? It clearly would not be equitable to pay the owner of the site $1,000,000 for the special value of his property, and yet the computation does not indicate any other sum.

The other class of evidence, as to such value, would be opinion evidence based upon knowledge acquired and sales made in the interval between the first investigations and the date of taking of the property, and this kind of evidence, although more plausible, should not, in the opinion of the committee and as already suggested, be used to create a special value for availability and adaptability when such value did not exist before the investigations were made.

The subject of the special value of land is referred to in Mr. Justice Hughes' opinion in the Minnesota Rate Cases. The discussion related to "the endeavor to apply the cost-of-reproduction method in determining the value of the right of way" (page 451):*

> It is urged that, in this view, the company would be bound to pay the "railway value" of the property. But, supposing the railroad to be obliterated and the lands to be held by others, the owner of each parcel would be entitled to receive on its condemnation its *fair market value* for all its available uses and purposes. If, in the case of any such owner, his property had a peculiar value or special adaptation for railroad purposes, that would be an element to be considered. But still the inquiry would be as to the fair market value of the property; as to what the owner had lost, and not what the taker had gained. The owner would not be entitled to demand payment of the amount which the property might be deemed worth to the company; or of an enhanced value by virtue of the purpose for which it was taken, or of an increase over its fair market value, by reason of any added value supposed to result from its combination with tracts acquired from others so as to make it a part of a continuous railroad right-of-way held in one ownership.

A recent opinion upholding an allowance for reservoir availability and adaptability is that of the United States Circuit Court of Appeals, decided July 15, 1913, and it relates to a parcel of land taken for the Ashokan Reservoir of the New York Water Supply.

> That the Ashokan site is peculiarly suitable for reservoir purposes cannot be disputed. Indeed, it may almost be said that it is the only available location for a reservoir from which the great City of New York can be supplied with an abundance of pure water. Located, as the city is, on a narrow peninsula, between two tidal rivers, it is evident that the choice of sites which the State can control is an exceedingly limited one. A glance at the map seems to demonstrate the proposition that the supply of water for such an immense number of people must come from a reservoir located west of the Hudson and above the New Jersey line. The Ashokan site could not escape the attention of a competent engineer employed to make the selection. The process of exclusion would inevitably bring him to the Esopus watershed. Its availability for furnishing New York with pure water was appreciated fourteen years ago, when the Ramapo Company was organized for the purpose of selling the water in question, not only to the City of New York, but to other cities of the State located on both banks of the Hudson. The availability of the Ashokan site induced the City of Kingston to make a careful examination of its capacity for furnishing a supply of water to that city. In short, without entering further into details, it can hardly be disputed that the Ashokan site was the natural place for the reservoir which is to supply the fast-increasing multitude of people who dwell on both sides of the Hudson, and that this availability had been proved and was publicly known long before the City of New York instituted these proceedings. It must have been evident to all intelligent land owners that their property would, in the near future, inevitably be acquired as part of an immense water system. That this demand increased the value of these lands follows as a necessary conclusion. To value them only according to the tons of hay or the bushels of potatoes they produce ignores the other element of value, namely, that their possession was necessary in order that water might be furnished to the increasing millions along the banks of the Hudson.

To refer to a minor feature of this opinion, it refers to the city of Kingston as having examined the Ashokan Reservoir site with reference to its capacity for furnishing a supply of water to that icty. Kingston had, in 1910, a population of 25,908. The estimated cost of the reservoir upon this site is upward of $30,000,000, making the cost per inhabitant of Kingston the prohibitive sum of $1,150, showing that a reservoir upon this site would be unavailable and unadapted to the use of Kingston on account of its cost, and how little weight should be given to such evidence that there was competition for such a property which would give it a special market value.

The Appellate Division of the Supreme Court (130 App. Div., 350, 356) and Court of Appeals (195 N. Y., 573) of New York State, passed upon a similar case (McGovern vs. City of New York) relating to the compensation for other parcels of land taken for the Ashokan Reservoir, and reached the opposite conclusion from that reached by the United States Circuit Court of Apepals.

The Appellate Division, referring to the action of the Commissioners making the award, said:

> Their duty was to award compensation for the taking of the land and not for the use to which it will be applied when taken.
>
> It is only when it is shown that it has a market value for some particular use that the availability and adaptability of the property to the use can be taken into account.
>
> It is true that he [the owner] is not limited in compensation to the use which he makes of his property but is entitled to a fair market value for any use to which it is adapted by virtue of its location and for which it is available * * * The value of property is not limited by the present use, or the use for which it is sought, as either may be more or less than its market value. For example, land may be valuable, abstractly considered, for reservoir purposes, but its market value would depend upon a demand for such a purpose. If no one desired the property for a reservoir, its value might be much less than for any other purpose * * * No evidence was given in the present case tending to show that before the land was taken by the city it was regarded as more valuable because of its advantage of location and adaptability for use as a reservoir.
>
> The appellant did not prove or attempt to prove that the value of the property in question, or any of the property included in the reservoir site, had been increased by its adaptability or availability for reservoir purposes before the commencement of this proceeding.

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*Simpson et al. v. Shepard, same v. Kennedy, same v. Shillaber, 230 U. S., 352, June 9, 1913.

There is no shadow of evidence of any prior demand for the property as a reservoir site or of any customer who would give more for it for that purpose, or of any circumstance by which the value of the parcel in question, as a part of a natural reservoir site, could be estimated or determined.

These cases went to the United States Supreme Court and were decided in June, 1913. The judgment of the lower court was sustained, but the points considered in this report were not treated at length. The court referred to "the difficulties in the way of such evidence (that relating to availability and adaptability) and the wide discretion allowed to the trial court" and added, "Much depends upon the circumstances of the particular case."

In the judgment of the committee, the enhancement of the value of land on account of its availability and adaptability for special purposes, in cases where the property is especially available and adapted only to the community or public utility acquiring it, is against sound public policy, and a just consideration of the right of the public to have public works created at a fair cost.

EXCESS OR DEFICIENCY OF PAST EARNINGS.

It is the view of the committee that an excess or deficiency of past earnings of an unregulated property should not be considered in making a valuation of the property, except as they may be useful in determining the development expenses. The committee has been led to reach this conclusion, not because it would necessarily be inequitable to consider such past earnings, and especially recent earnings, but there seems to be no rational basis upon which they can be included. They are important as indicating whether the rates are too high or too low.

If one attempts to consider past earnings in making a valuation, many questions arise. Shall the consideration include the earnings for 2, 5, or 10 years, and if so, why not for a much longer period? If the earnings have been more or less than a fair return upon the capital employed added to a proper sum to cover obsolescence and depreciation of all items of property, is the corporation accountable to the public for the surplus or deficiency, as the case may be?

From the time that a public service commission is authorized to control rates, to fix the amount that the corporation may earn as a depreciation allowance, and to control methods of accounting, it is desirable and proper to take into account the excess or deficiency of earnings. The public service corporation should be required to account for its surplus, and, on the other hand, due consideration should be given in fixing rates to any deficiency which may occur.

The subject has been very well discussed by Commissioner Prouty of the Interstate Commerce Commission in Spokane vs. Northern Pacific.*

Referring to a surplus which had been accumulated and invested in the property, he states:

> Should the Government today take note of that surplus for the purpose either of so reducing the rates of the company that no earnings can be made upon this much of the property or with a view to in some sense turn that surplus back again into the hands of the public?
>
> * * * During all this period the excess has gone into the property, which has gradually become more valuable, and this increased value has reflected itself in the market price of the securities of that company. It is impossible to restore what has been improperly taken in the way of excessive rates to those persons from whom it has been received. The Government, under those circumstances, cannot lay hold on this surplus as a fund held in trust for the public.

This case strongly illustrates the fact that if any Government tribunal is to do justice between the railway and the public, if it is to feel any confidence in the correctness of its conclusions, its supervision must be continuous and not spasmodic. There must be some point of departure and from that point the knowledge of the Government must be accurate and complete. After earnings have once been "capitalized" and benefits have been "conferred," when the various independent organizations have been perfected, it is impossible to either know or to undo.

ADAPTATION AND SOLIDIFICATION.

When items of property are first ready for use or for operation, they are generally incomplete. If the item of property is a machine, it requires testing and minor changes or sometimes the replacement of parts before it will operate smoothly. A railroad roadbed, built of earth that has been dumped without consolidation, will, as the result of running trains and the effect of weather conditions, settle, and earth will be washed from the fresh slopes, so that additional earth, ballast and labor will be required to produce a solid and full-sized embankment. All such costs required to complete machines and structures are properly chargeable to the capital account, and if the unit prices used in making the valuation do not cover these items, they should be added.

If a valuation is to be based on book accounts, it is equally proper to include these items in the valuation, provided duplication is avoided. If the accounts are so kept that these expenditures have been charged to operation, swelling the operating and diminishing the capital account, then they would appear as development expenses, because the deficiency of earnings in the early years of the operation of the property would be greater than if they had been properly charged to the capital account.

Embankments and the slopes of cuttings may become more stable on account of the growth of grass and weeds, and may therefore be considered to be more valuable, but such value as this, which is not the result of expenditure by the corporation, should have no place in the valuation of the property.

SHOULD RATES BE BASED ON PHYSICAL VALUE ONLY?

There are several reasons why a physical valuation, taken by itself, furnishes an unsatisfactory basis for determining rates, and other reasons why it does not seem equitable to omit other considerations.

The method of valuation recommended by the committee is (with some exceptions) based upon the cost of reproduction of the identical existing features of the property. It protects the actual investment in the property, so far as this can be done where a deduction is made for the depreciation of the existing perishable property and where present instead of original prices are used. The result is a physical valuation dealing with the cost of the existing structures, with little regard for merit or efficiency.

Efficiency Should Be Recognized.—Engineers probably recognize much better than others that the intrinsic value of any property is not always proportionate to its cost. One property may have been exceptionally well designed and located and be of such a character that it can be operated at comparatively small cost and give a high grade of service, while another property, having the same physical valuation, may have the opposite characteristics, being poorly designed and located, expensive in operation, and furnishing unsatisfactory service.

If the general principle, a fair return on the fair value of the property, is to be construed as a fixed percentage on the physical value of the property, the inferior service would be paid for at a higher rate than the better service. To avoid this result, some modification of the physical valuation or of the rate of return upon such valuation should be made by the rate-making body, with a view to obtaining more equitable results.

Although the committee recognizes that sound public policy requires a suitable reward for efficiency, it has been unable up to this time to formulate general rules which recognize this feature, and commends its special consideration to Public Service Commissions and to the courts in the cases coming before them.

Boston Consolidated Gas Company Sliding Scale.—A method of recognizing efficiency was put into practical use in Massachusetts in 1906, by a legislative act* which provides that the Boston Consolidated Gas Co. shall sell gas at a "standard price" of 90 cents per 1,000 cu. ft. and that the company may pay its stockholders a "standard rate" of 7 per cent per annum on the par value of its stock. If during any year the price charged by the company is less than the "standard price," the company may, during the following year, increase its dividends above the "standard rate" one-fifth of 1 per cent for every cent of reduction in the price of gas below the standard.

Other parts of the act provide for maintaining a surplus fund and limiting its rate of increase and total amount; for issuing, with the approval of the public service body, such additional stock as may be necessary at its market value; for reporting each year the cost of manufacture, including depreciation, and for readjusting the "standard price" at any time after 10 years from the date of the original act.

Under the provisions of this act, gas is now sold for 80 cents per 1,000 cu. ft., and the company pays a 9 per cent dividend to its stockholders.

Should a Higher Return Be Made Upon an Old Favorably Located Property?—It is frequently the case that an old property which has been in existence, say, 30 years or more, is so favorably located that it has, at apparently reasonable rates, rendered satisfactory service and paid continuously for upward of 20 years more than average dividends upon the stock. As instances of this kind of property may be cited railroads between important terminals, water works having a supply from nearby and adequate sources, and gas works in growing cities and towns. As a result, the stock of such companies has been sold far above par and the investment in such stock has not been regarded as speculative because the investors had every reason to suppose that the existing conditions would be likely to continue indefinitely.

The question then arises whether the failure to regulate the rates of such property for a long term of years has not in effect warranted the assumption, made by those investing in the property, that rates producing the same liberal return would continue; also whether as a matter of equity the rates should not in the future be fixed on a basis producing a more liberal return than on properties which, by reason of poorer location, although charging higher rates and furnishing less satisfactory service, earn smaller returns than the more favorably located property.

It has been held by the committee that the unfavorably located public service property is not entitled to more than reasonable rates for the service it renders even though the property in question becomes a losing venture; conversely, the committee believes that the favorably located old property should equitably receive liberal treatment by rate-making bodies.

CONTINUOUS REGULATIONS BY COMMISSIONS.

The committee has, in several instances, called attention to the difficulty of carrying out certain equitable and desirable features of valuation and rate regulation where the control of such matters is spasmodic, and not based upon continuous commission control in connection with legislation adapted to obtaining the desired results.

Some of the defective features of spasmodic regulation without adequate laws are:

1.—Valuations vary from time to time with changes in the prices of labor, materials, and land, involving undesirable gain or loss to the investor or the rate-payer, as the case may be, and increasing the hazard of the investor and consequently the rate of interest.

2.—It is impracticable to take account of an excess or deficiency of past earnings, because, if the public service corporation is allowed to charge rates which result in the accumulation of a surplus or deficiency, neither the corporation nor the public can be compelled subsequently to account one to the other therefor.

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*Spokane v. Northern Pacific Railway Company, 15 I. C. C. R., 376, 415, February 9, 1909.

*Chapter 122, Acts, 1906.

3.—The prevailing lack of method in determining depreciation allowances may result in earnings by the corporation of much more or less on this account during the life of each item of property than its value, and a similar difficulty is encountered in properly accounting for the appreciation of land and other property which increases in value.

Where it is feasible to obtain suitable legislation and the appointment of intelligent and independent commissions governed in their actions by considerations of equity, uniformly fair relations between the investors and the public can be maintained much more evenly than under present conditions. Accounting under such control would have a more important place than at present.

Under such control the valuation of a new property would be based mainly upon book accounts with such adjustments as might be necessary for the value of services rendered and not paid for; of risks incurred, if the commission acting for the public would not assume the cost of accidents and failures, and of the use of money upon which interest was not actually paid. The cost of developing the business could also be determined on the basis of the accounts.

Allowances for the depreciation in the value of perishable property and for the appreciation in the value of land could be made on a rational basis and charged periodically to the capital account. If the net depreciation allowance in any year or series of years were made too small or too large, the effect of the error would be diminished because the investor would have, respectively, a larger or smaller remaining capital entitled to an annual return. If after a series of years the depreciation allowances should prove to be more or less than the actual depreciation of the property, the allowances could be increased or diminished to provide a proper adjustment.

A good-sized surplus is desirable for preventing frequent changes in rates and dividends, and for producing stable conditions. The gradual creation of such a surplus would be warranted under continuous commission control if it were to be held for the benefit of the public when not needed for maintaining authorized uniform dividends.

SUMMARY.

1.—Report Limited.—This report is limited to the discussion of valuation for the purpose of rate-making. Valuations for other purposes differ in some respects, for legal and other reasons.

2.—General Principles. — The valuation should be such as to lead to equity as between the owner on the one hand and the public on the other. The rates should yield a fair return upon the fair value of the property. The owner, in addition to a reimbursement of the current expenses of operation, is entitled *(a)* to an annual return, covering interest and profit, for the use of his capital, having in view the risks incidental to the investment, and *(b)* an annual allowance to provide for the net depreciation in value of all the items of physical property, resulting from all causes, this allowance to be sufficient to pay for all items of property at the time they cease to have value. As earnings are based to a large extent upon rates, they cannot serve as a basis of valuation for rate-making.

3.—Method of Determining Physical Value.—The committee recommends the use of the actual reasonable cost, subject to suitable provisions for the cost of developing the business, for depreciation, etc., as the best basis for valuation of a new or recently created property, and in the case of an old property, the estimated cost of reproducing the property, with similar adjustments. As the actual cost of a new property should be substantially the same as the fairly estimated cost of reproduction of such a property, this is in effect a recommendation of a practically uniform method of valuation.

4.—Future Properties Under Continuous Commission Control and suitable legislation could be valued best on the basis of actual reasonable cost, using book accounts for the ascertainment of the cost of developing the business and making proper charges in such accounts for depreciation. Under this method the corporation would avoid the hazard due to the rise and fall in prices.

5.—Identical Plant Should Be Valued.—The valuation should be predicated upon the identical existing plant, and not upon a substitute plant capable of furnishing equally efficient service.

6.—Unused Property.—The courts have indicated that the property to be considered in making a valuation for rate-making should be only that devoted to the public use. A commission may or may not permit the inclusion of a part or the whole of the unused property in the valuation, depending upon the basis adopted by it.

7.—Reproduction Should Be Under Original Rather Than Present Conditions.—The valuation should be based on the conditions existing at the time the various portions of the property were built, but on the prices prevailing at or near the time of the valuation. As the result of the adoption of this view, the cost of pavements laid over water pipes after their installation would not be included in the valuation, but the extra cost of piecemeal construction would be included. Ownership would not at all times control as to the property to be valued; for instance, a railroad built along the margins of a reservoir by a water works corporation as a substitute for one occupying the site of the reservoir would be valued, while service pipes belonging to the corporation, if laid at the expense of the rate-payers, would not be valued.

8.—Inventory.—The inventory of the property should be made in detail and include an allowance for omissions varying in amount with the care and completeness with which it has been made. Due allowance should be made for the amount by which the actual overrun the theoretical quantities.

9.—Unit Prices prevailing at or near the date of the valuation should be used rather than original prices, but to make a valuation more stable, the normal or average prices prevailing during a period of from five to ten years prior to the date of the valuation should be used, except where the prices are steadily rising or falling, when prices nearer the date should be used.

10.—Overhead Charges should include the indirect and incidental expenses not covered by the unit prices used; preliminary expenses, incidental to investigation, promotion and report; commission for sale of securities; incidental expenses during construction, including engineering, inspection and other fees, administration charges, general expenses, contingencies, and interest and taxes during construction. Such charges have frequently been underestimated.

11.—Development Expenses should be included in the valuation. They have sometimes heretofore been referred to as "Going Value" when this term has been defined as the investment necessary to put the plant into successful operation and to create revenues that justify its construction. They have also been defined as the difference between the actual return and a fair return upon the fair value of the property during the early years of the operation of the original plant and of subsequent additions. The development expenses connected with ordinary additions to a plant are a much smaller percentage than those connected with the original plant.

12.—Working Capital.—A reasonable sum should be included in the valuation for working capital.

13.—Depreciation Allowance.—The corporation is entitled to an annual allowance sufficient to pay for all items of property at the time they cease to have value.

14.—The Depreciated Value of Property Should Be Basis for Rate-making.—The depreciation allowance is in effect a re-payment in instalments from the rate-payer to the corporation of a part of its investment in particular items of property, and the sums repaid should not thereafter appear in the valuation of such items.

15.—Expectation of Life is a necessary factor in determining depreciation allowances to pay for items of property during their life.

16.—Equal-Annual-Payment Method of Determining Depreciation Allowances. — The most desirable method is one which shall distribute the combined annual charges on account of the plant as equally as practicable among the rate-payers of different years; this result is reached by the Equal-Annual-Payment Method recommended by the committee. By it the depreciation allowances for any given item of property are so adjusted from year to year that, when taken in connection with the amount which the rate-payer must contribute on account of such item for a fair return on the remaining investment therein, and for repairs and operation thereon, they are substantially equal in each year of the life of such item. The method is illustrated by Table V and, to aid in its application, extended depreciation tables are appended.

Under this method the depreciation allowances are credited to the capital account, and not distributed as dividends, and replacements becomes charges against capital. The amount paid to the corporation annually for depreciation allowances would be used for additions, replacements, betterments, or the extinguishment of outstanding obligations, and when not needed for these purposes, would temporarily increase the working capital; the investment would be kept intact and earning a return on the full amount invested. This method conforms to the decisions of the highest courts.

17.—The Sinking-Fund Method assumes that returns are to be based on the full value of an item of property, and that uniform annual payments will be made to a sinking fund which is to be held intact with accretions of interest until the end of the life of the item in question. This method does not meet the requirements of the courts in that they object to the establishment of sinking funds by public service corporations, and the highest court has decided that returns must be based upon the depreciated and not the full value of the property. Sinking fund computations are used to determine the depreciation allowances of the Equal-Annual-Payment Method, the allowances for each year being equal to the sinking fund payment added to the interest on the accumulated fund at the beginning of the year.

18.—Straight-Line, Actual-Inspection, and Replacement Methods of Determining Depreciation.—The Straight-Line Method in the case of long-lived property distributes the burden very unequally among the rate-payers of different years, as will be seen by Table VI. The Actual-Inspection Method does not determine the amount of the annual depreciation allowance which the corporation is entitled to earn, and is likely to be unfair. The Replacement Method does not provide for paying for items of property during their life, and is especially unfair in the case of long-lived items of property.

19.—Appreciation.—Land and water rights generally appreciate in value from year to year. The committee recommends that such appreciation be treated in the same way that the depreciation of the perishable property is treated, but on the opposite side of the account. Under present practice the appreciation generally inures to the benefit of the corporation, and it may not be feasible to carry out the committee's recommendation except under necessary legal authority and continuous commission control.

20.—Method of Valuing Land.—Land should be valued at present rather than original prices. On account of the difficulty of correctly appraising the value of public service land and of recent decisions of the highest court, land which has been purchased at a fairly recent date should be valued on the basis of its cost with additions proportionate to the increase in the value of neighboring land not affected by the use to which the public service land is devoted. When it is not practicable to use the cost as a basis, land can be valued most correctly on the basis of present market value of neighboring land of similar character, augmented by the ratio ordinarily found to obtain in that region between land acquired by public service corporations, on the one hand, and by private parties on the other, and subject to the value of the improvements upon the land obtaining at the time of purchase. Those valuing land in cases coming before the courts should pay due regard to the decision in the Minnesota rate cases fully discussed in this report.

21.—Past Earnings.—It is the view of the committee that an excess or deficiency of past earnings of an unregulated property should not be considered in making a valuation, except as may be useful in determining development expenses; but, under continuous commission control and proper legislation, it would be desirable to take such earnings into account from the beginning of such control.

22.—Adaptation and Solidification should be included in valuation to the extent they may have involved the expenditure of money. The increased value due to adaptation and solidification which has resulted from the action of the elements and natural causes without the expenditure of money should not be so included.

23.—Efficiency Should Be Rewarded.—A physical valuation, dealing with the cost of existing structures with little regard to their merit or efficiency, is not by itself a satisfactory basis for rate-making, because it does not offer the reward for efficiency which sound public policy requires. The committee believes that some incentive, such as a share of the profits or savings resulting from special efficiency, should be given to the corporation, but has not been able up to this time to formulate general rules for providing such reward.

(The End.)

Course in Highway Engineering, Kansas State Agricultural College.—The Board of Administration of the Kansas State Agricultural College, Manhattan, Kansas, has established an engineering course in highway engineering to begin next year. The course is offered as an option for students in civil engineering. The college year is divided into three terms. Five hours of strictly highway work will be given the second term of the sophomore year; ten hours the first term, four hours the second term, and ten hours the third term of the senior year. W. S. Gearhart, state engineer, has been appointed professor of highway engineering in charge. Most of the work will be given by L. E. Conrad, professor of civil engineering, but a part of the practical subjects will be handled by Mr. Gearhart and his assistants. The board also established a ten weeks' short course in highway engineering for county engineers and surveyors, to be given during December, January and February.

Comments on Methods of Computing Earthwork Overhaul.

To the Editors: In your issue of Jan. 14, 1914, you describe a method of computing overhaul, as given by Mr. Frank Gahrtz. I wish to make a few comments upon the subject and to submit a method which I have frequently seen in use.

It is a fact that overhaul computations are often made incorrectly as shown by the comparison of methods given by the American Railway Engineering Association and by Molitor and Beard's "Manual for Resident Engineers." The error in the two methods named arises from the assumption that the overhaul diagram is a trapezoid, which is seldom the case. But these methods give correct results when the diagram is divided into any convenient number of trapezoids, as shown by Table I and by Fig. 1.

The result as shown (11,370 units) differs from Mr. Meier's figures by only 360 units, certainly a much more allowable error than that of Molitor and Beard's method (3,730 units too small), or the American Railway Engineering Association method, 810 units in error. It can readily be seen that large mistakes can be made by using either of these two named methods. A little study of the diagram on the part of the computor, and consideration of the fact that overhaul is represented by an area, would eliminate serious mistakes of the kind mentioned.

TABLE I.

Ft. hauled From	Ft. hauled To	Aver. dist., ft.	O. H. stas.	Cu. yds.	Units of overhaul.
500	600	550	0.50	600	300
600	700	650	1.50	550	825
700	800	750	2.50	550	1,375
800	900	850	3.50	450	1,575
900	1,000	950	4.50	400	1,800
1,000	1,100	1,050	5.50	300	1,650
1,100	1,200	1,150	6.50	300	1,950
1,200	1,315	1,257.5	7.575	250	1,895
Total			11,370	cu. yds. 3,400	O. H.
Free				1,800	
Check				5,200	

On all approximate estimates the writer uses a method by areas. The case under consideration may be illustrated as follows: By the aid of a planimeter the area, below the 500-ft. "freehaul line," is found to be 3.55 square inches. Each square inch represents 8,000 "yard-stations" (4 stations hor. by 2,000 yards ver.). Thus the total area (3.55) multiplied by 8,000 will give 2,840,000 "yard stations." This amount divided by 3,400 yards (ordinate between 1,800 yds. and 5,200 yds.) gives average haul distance, 8.353 stations. Deduct free haul of 5 stations, leaving "overhaul distance" of 3.353 stations. Thirty-four hundred yards hauled 3.353 stations equals 11,390 units of overhaul—a pretty close approximation for planimeter work. The operation should not take more than five minutes once the diagram is constructed, regardless of size of diagram or irregularity of outline.

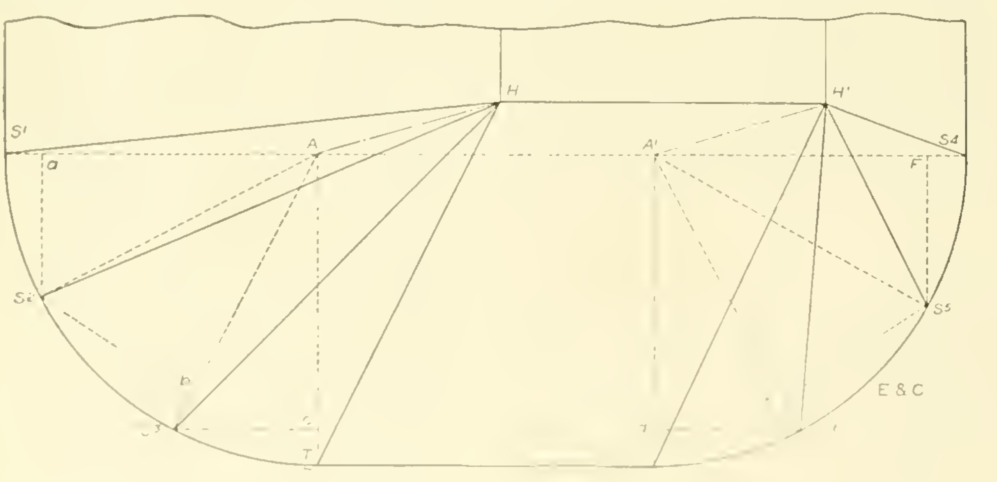


Fig. 1.

It is surprising to notice the number of slipshod methods used in computing overhaul, and the excuses advanced for them, such as the following: "Overhaul only costs 1 cent per yard," "Further approximations are not warranted by the nature of original cross-sections," and others.

Hints for plotting diagrams, and overhaul definitions may be found in the American Civil Engineer's Pocket Book, page 142.

Very truly yours,
F. T. Howes,
Assist Engr. Oliver Iron Mining Co.
Hibbing, Minn., Jan. 20, 1914.

To the Editors: With regard to the methods of calculating overhaul described in your issue of Jan. 14, 1914, the writer would like to point out that Gahrtz's Method III will on examination be found on the whole less favorable to the contractor than Method II, as unless the material is hauled in only one direction from a cut Method II favors the contractor.

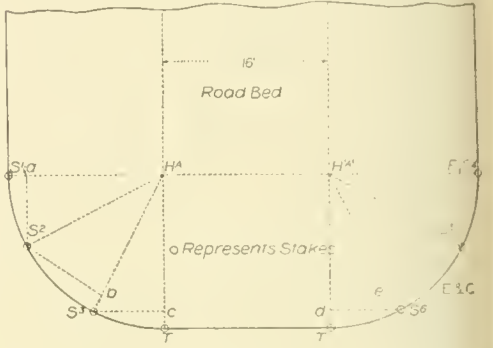


Fig. 2.

It is obvious that Method III is mathematically correct though rather cumbersome. However, in all cases a simple and exact graphical solution is available, based on the fact that the total number of equivalent yards of overhaul is equal to the shaded area under the haul curve and outside the freehaul limits, as shown by Fig. 2. The writer would however, like to point out that in many cases the use of Method II may have been specified or implied before the contract was let, and that unless Method II happens to work a hardship on the contractor the use of any other method is hardly legitimate.

Very truly yours,
J. R. Wade
City Engineer's Office, Waterloo, Ia.
Jan. 28, 1914.

Use of Stadia Hairs for Running Grades.—Stadia wires may be used for running grade lines. Thus when the telescope is level the upper and lower wires give + 0.5 per cent, and — 0.5 per cent grades running of course from the telescope axis. If the telescope axis is 4 ft. above the ground; and points on the ground at a distance that would be reached by a + 0.5 per cent grade are desired, the rodman must set the target at 4 ft. on his rod. All points on which the rod is held where the target shows behind the upper stadia wire are on a + 0.5 per cent grade from the site of the instrument. Also, after first noting a distant object covered by the middle wire when the telescope is level, by moving the telescope vertically until the top or bottom wire covers the same object the other wire will cover objects on a + 1.0 per cent or a — 1.0 per cent grade. By similar combinations 1.5 and 2.0 per cent grades may be run. Grades of intermediate per cents may be run approximately by this method by estimation.

Roads in Ohio.—The various kinds of roads in Ohio, amounting to a total of 81,299 miles, are divided as follows: Brick, 500, concrete, 66; earth, 54,242; gravel, 11,944; macadam, 11,403; miscellaneous, 74.

Engineering and Contracting

Devoted to the Economics of Civil Engineering Design
and to Methods and Cost of Construction

Volume XLI. CHICAGO, ILL., APRIL 22, 1914. Number 16.

An Interesting Court Decision on "Development Cost" and "Unearned Increment."

Some time ago the Supreme Court of Oklahoma rendered a decision in which the "Wisconsin method of determining going value" was favorably considered. But the first eastern state court decision in which the "Wisconsin method" is given approval has just been rendered by the Supreme Court of New York, in a case involving the rates for gas charged by the Kings County Lighting Co., of Brooklyn. The full text of the decision will be given in our next issue.

The most important part of the decision relates to "going value" or "development cost." The court holds that "going value" must be determined and separately set forth by the public service commission. The commission can not hereafter say that it "has given the matter consideration" without stating definitely the dollars and cents result of that consideration. This last is unquestionably a rational ruling. The court holds that the accrued deficit in fair return constitutes the development cost or "going value." While this is beyond question true, we think the court has failed to note that this theory of arriving at development cost does not accord with the "cost of reproduction theory" of appraising property. It does not even accord logically with the court's decision that the company is entitled to have all increment in land treated as an additional profit and not as an ordinary income. In brief, the court has failed to see that the "development cost" theory is part and parcel of the agency theory of public utility control, which agency theory is a relatively new theory that cannot be made entirely retroactive without leading both to injustice and inconsistency. The "Wisconsin method" of determining development cost is a logical deduction from the agency theory, but courts and commissions usually fail to ask by what right a company can be forced to make an accounting today for all its previous acts when in truth it was not an agent in the present sense of that term. We speak in one breath of the legal principle that forbids the enforcement of retroactive laws, and in the next breath we adopt a theory that virtually makes every utility commission law retroactive in the fullest sense. This is a serious matter, and one that has never been presented squarely to any court. What is a sound law for the present and future is not necessarily a law that can be appealed to for the past.

A careful reading of the decision under discussion discloses the fact that, while the court favors the "Wisconsin method," it does not declare it to be the only method that may be used in estimating "going value." The court says: "It is proper for this court, however, to indicate a permissible method or methods." And the court comments not unfavorably upon the method of estimating the cost of building up the existing business in a hypothetical period of time, which is a method that goes logically with the cost of reproduction theory.

The court refuses to follow the U. S. Supreme Court decision in the Consolidated Gas case, as to allowing the cost of pavement laid over mains but not paid for by the company. Here again the court follows the agency theory. But it immediately abandons the agency theory when it refuses to regard increment in land value as income. The court says:

> We agree with the Appellate Division that annual increase in the value of land is not income. Of course, under the rules of the Ames case, land might become so valuable as to require its use for other purposes and as to make a rate based on it unfair to the public. The present is not such a case.

We repeat what we have had occasion to say several times in the recent past, that until commissions and courts see clearly that there are at least two theories of rate control there can be no consistency in the various decisions involving appraisals and rates. There are two clearly separate theories: (1) The Agency Theory and (2) the Competitive Theory. There is a third that is a mixture of the two, and it is this third theory that has actually prevailed for many years, and is still in active existence even in such states as Wisconsin. In some measure public service companies have been treated as public agents and in some measure they have not. Insofar as they exercise powers of eminent domain they are public agents. But insofar as they are forced to engage in cut-throat competition with other companies, and, indeed, with municipally owned plants, they are not public agents.

It is essential to get back of the mere words that state our theories, and ascertain the facts that do or do not justify the application of a given theory. It is essential to distinguish between what is a fair and legal theory to apply in the future and what is a fair and legal theory to be used in interpreting the past.

The Need of Better Designed Erection Equipment.

Occasionally there is brought to the attention of the engineer an account of some "accident" in connection with the erection of some structure—perhaps a cable has broken, or a chain has failed, or such a little thing as a hook has straightened out or fractured. It is quite common practice to refer to such occurrences as "accidents" instead of as "failures," intimating thereby that they are beyond control and are to be expected on construction work. If no one is killed or seriously injured, mention of such failures seldom gets beyond those in direct charge. It is unquestionably true that the number of these failures could be greatly reduced if engineers would give more attention to construction equipment. Moreover, much of the success of a completed project is due to a correctly designed and well balanced erection equipment. On account of the apparent simplicity of such equipment engineers and contractors are somewhat inclined not to give its design serious consideration, but are content to accept the manufacturer's product without even checking its rated capacity. Too often is it true that the builders of erection devices forget that it is necessary to keep in mind the principles of mechanics and to be governed by the working strength of various materials. It must be remembered that erection equipment is not ordinarily used under the most favorable conditions, nor is it always operated by men who appreciate its limitations. In the construction of large bridges and special structures we are beginning to realize that the design of the erection equipment is a part of the engineer's work and should not be left entirely to the manufacturer or even to the contractor. In ordinary bridge and building construction, however, such equipment is not given sufficient attention. It is not difficult to determine the strength of various parts of construction devices, yet we believe that engineers in general are not well informed on this subject. Those who have made a study of erection equipment realize that under existing conditions it is essential that low working stresses be used; yet an inspection of the construction details of such equipment as derricks and cranes, where impact is to be expected, will show that the unit stress is frequently as high as the elastic limit of the material.

In this issue we are publishing a paper which treats of the strength of equipment for handling loads. Formulas are given for determining the strength of hooks, rings and chains, and the strength of ropes and cables is also considered. Various details of erection equipment are discussed, and some of the common defects of such equipment are pointed out. The working stresses recommended by the author are considerably lower than those in common use, but there is need for large factors of safety in the various parts of erection devices. Although the subject is by no means treated exhaustively, some valuable data are given, and the author is to be commended for having formally brought to the attention of engineers the need of better designed erection equipment.

Should Some Failures of Bituminous Macadam Built by the Penetration Method Cause Its Abandonment?

If the answer to the above question be affirmatively given simply because of "some failures," every type of road in existence would be abandoned for the same reason—"some failures." When, however, any type of road shows a large percentage of entire or partial failure in the hands of competent road builders, then, and not till then, should that type of road be regarded with great suspicion. Judging road types by this criterion we fail to be in accord with the recommendation of the consulting engineers who advocated the discontinuance of road construction by the penetration method in New York state.

There have been many instances cited pro and con to indicate the good and bad features of roads built by pouring bitumen into the voids of broken stone after it has been spread upon the subgrade. We shall select but one example, yet on that one it would be safe to risk the entire argument in favor of continuing the use of the penetration method. The example we select is the road system of Los Angeles county, California. The large mileage of asphalt-macadam in that county, and the fact that the "mixing method" has not been used there at all serve to make this example worthy of careful study.

Some of the earlier asphalt macadam roads built by the county went to pieces, or, to express the facts more precisely, rutted badly under traffic. Upon digging up the rutted portions, it was found that an excess of asphaltic oil had been used in some cases, and in nearly all cases it was apparent that the broken stone had not been sufficiently rolled before pouring the oil over the surface. As a result of this experience, the specifications were so changed as to provide a water-bound macadam base with a thin top of asphalt-macadam made by the penetration process. Roads thus made have been in existence several years under exceedingly dense traffic, and have given satisfaction. In extensive traveling, the writer has nowhere seen as large a mileage of fine roads in the same area as may be seen in Los Angeles county.

In this connection it may not be amiss to remark that eastern engineers are far too apt to attribute western successes in road building to "climate." A poor bituminous road, however, will fail in any climate.

California, it should be remembered, was the state that pioneered the penetration method of bituminous macadam construction. Indeed the penetration method, as it is now called, was itself an evolution from the surface sprinkling of "dirt roads" with asphaltic oil, the next step in that evolution being the sprinkling of asphaltic oil upon successively applied layers of gravel. In view of the history of the penetration method it is rather remarkable that so many eastern highway engineers have failed to visit California, and seem to have given so little weight to the descriptions of satisfactory roads built in that state.

Water Works

Construction Plant and Methods Employed in Building a Reinforced Concrete Standpipe, 100 Ft. High and 40 Ft. in Diameter, at Fulton, N. Y.

The construction of the reinforced concrete standpipe at Fulton, N. Y., was begun on Oct. 1, 1913. In that latitude the winters are severe, so that speed in construction was requisite owing to the nearness of the winter season. Realizing this fact, those in charge of construction operations planned every detail of the work with a view to rapid construction. Good construction was of course also essential. Special precautions were taken to insure the securing of a seamless watertight cylinder. The construction plant and methods employed are here described from information contributed to the "Concrete-Cement Age" by Mr. Allen D. Whipple, construction engineer with the A. Monsted Co. of Milwaukee, Wis. The copy for illustrations shown herewith was made available for our use through the courtesy of the journal named.

The Fulton standpipe is of the slip joint type and is 40 ft. in inside diameter and 100 ft. high. The use of a radial truss within the standpipe, which was used as a support for the inside working platform during concreting operations and was ultimately used to support the dome forms for the concrete roof, was the distinctive construction method employed on this job. This truss was, of course, gradually raised from the bottom to the top of the standpipe as hereinafter described.

Handling the Centering.—Two methods of handling the falsework were permitted under the specifications. If seamless work was to be realized, the method of building two or more sets of forms designed to be set up one on top of the other and filled successively could not be considered. The time which would necessarily elapse between the filling of one set of forms and the next would be several hours under the best of conditions, and the setting process of the cement would have progressed so far that a bonded joint would have been required at each setting. While it is not certain that these planes of separation would have leaked, it is Mr. Whipple's opinion that, with the class of labor available, there would have been great probability of such leakage. As the stresses ordinarily set up in such walls tend to produce horizontal cracks, the presence of these bonded planes would be an element of worry and danger and might even negative the demonstration of the theory involved.

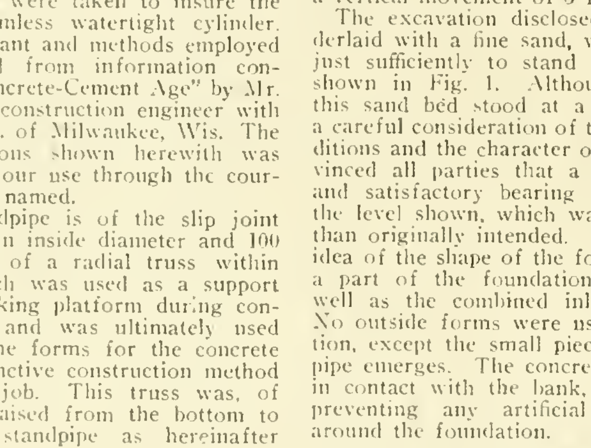


Fig. 1. View of Completed Excavation for Reinforced Concrete Standpipe at Fulton, N. Y., Showing Foundation Reinforcement and Combined Inlet and Outlet Pipe in Place, and Concreting Plant.

Something like the foregoing consideration led the contractor to believe that the system of gradually and continuously sliding the form work up the comparatively green wall, keeping the forms continually filled with fresh concrete, was the ideal method to adopt. This was accordingly done. The movement of the forms was so slow and gradual that an onlooker would not have readily noticed that the forms were moving at all; still they averaged a vertical movement of 5 ft. per day.

The excavation disclosed a gravel bed underlaid with a fine sand, which was cemented just sufficiently to stand up in the bank, as shown in Fig. 1. Although the surface of this sand bed stood at a considerable angle, a careful consideration of the surrounding conditions and the character of the bed itself convinced all parties that a practically uniform and satisfactory bearing had been found at the level shown, which was somewhat deeper than originally intended. Fig. 1 gives a good idea of the shape of the foundation and shows a part of the foundation steel in place, as well as the combined inlet and outlet pipe. No outside forms were used for the foundation, except the small piece shown where the pipe emerges. The concrete was cast directly in contact with the bank, with the desire of preventing any artificial drainage surface around the foundation.

Lighting for Night Work.—Electric lights were installed to furnish light for the night work and proved to be quite satisfactory. They were disposed in clusters on poles, one group on the post, over the inlet, another at the corner of the cement shed and another at the corner of the office building in the middle distance, and in other locations not shown in this view. Carbon filament lamps were used where they were liable to rough treatment or excessive vibration, and tungsten lamps were used for the more solid and protected positions.

The concrete was deposited through spouts, suspended from a cable attached to the tower at one end and made fast to a convenient tree at the other; the upper spout was open, being shaped like the letter U and provided with angles at the upper edges to act as stiffeners. This arrangement served to convey practically all of the concrete used on the entire job, the cable and spout being raised up the tower as the job progressed.

CONSTRUCTION PLANT.

The plant used was as follows: An erecting tower of timber was built as a first operation to a height of 164 ft. in order that there should be no necessity of handling timber over the heads of concrete laborers working at the base of the tower. Particular attention is called to this tower. It was 6 ft. x 12 ft. in plan and was provided with a complete stairway within the structure from bottom to top, for the use of the oiler who cared for the running gear at the top the hoist, for the use of a man at the hopper who regulated the flow of concrete in the chutes and for all laborers engaged on the tank in going to and from their work. Experience has proven that it is economy in the long run to build an expensive tower. The time saved by the men in going to and from their work repays the extra cost, not to mention the increased rigidity of the tower, and, greatest of all, the prevention of accidents.

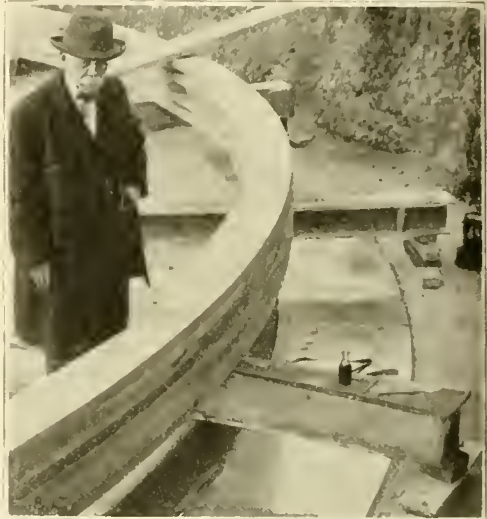


Fig. 2. View Showing the Placing of the Graphite and Sheet Copper Slip Joint at Base of Shell of Fulton, N. Y., Reinforced Concrete Standpipe.

The plant further consisted of a Smith concrete bucket hoist, the rails of which are mounted outside the tower on the side opposite or furthest removed from the standpipe. This bucket was raised by a "single lead" cable in conjunction with a Lidgerwood hoisting engine, as shown in Fig. 1. An additional 30 HP boiler was installed near the hoisting engine to furnish steam for heating water and such other purposes as might be required.

The mixer selected for this job was a Smith "No. 101 Mascot." This little mixer performed all the work required of it fully up to the contractor's expectations.

The total yardage of the shell for this standpipe is very small, approximately 7½ cu. yds. per foot of height, and, as Mr. Whipple is particularly insistent that concrete be not allowed to remain over five minutes in a revolving mixer, after the water is added, a very small capacity per batch was indicated, for the reason that possible accidental or unexected difficulties in handling steel, etc., might nake it impossible to receive a large batch immediately, while a small batch could almost lways be accommodated in the wall. This bviates the necessity of holding the batch

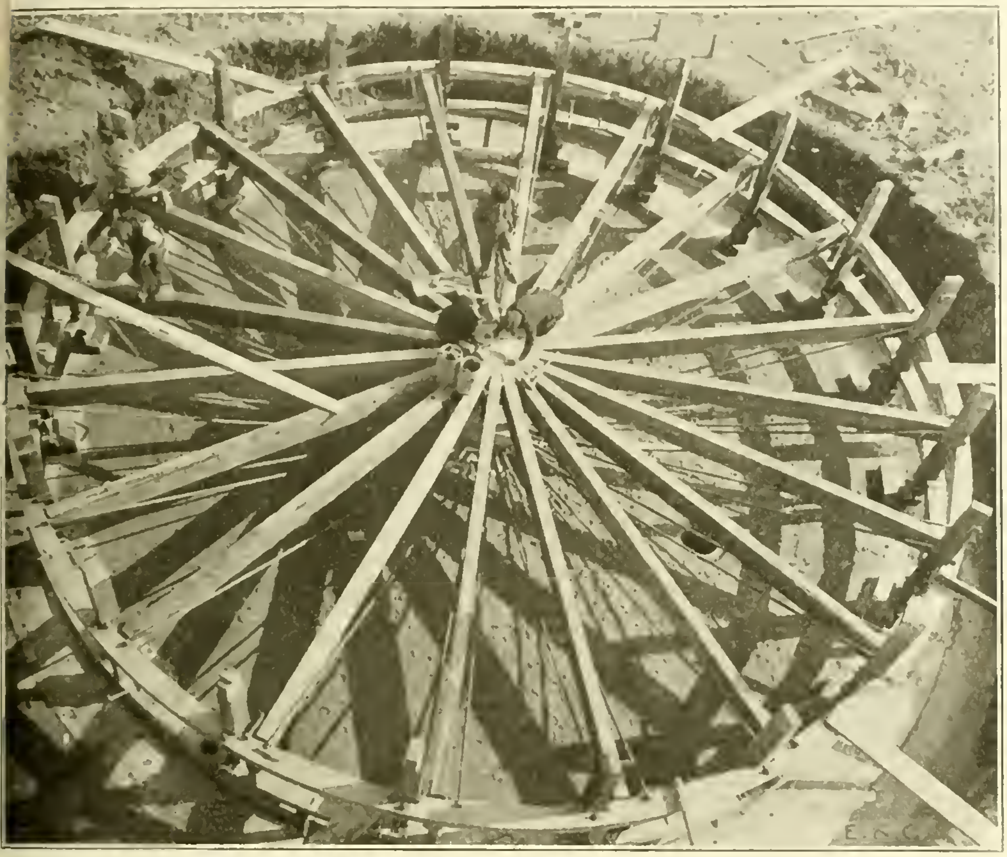


Fig. 3. View, from Concreting Tower, of Movable Radial Truss for Bracing Inside Form and for Supporting Working Platform and Roof Forms of Fulton, N. Y., Standpipe.

in the revolving mixer, or worse yet, depositing it in the bucket or hopper during an enforced wait. In other words, the contractor's desire was to produce, as nearly as possible with a batch mixer, a continuous flow of freshly mixed concrete, comparable in volume to the yardage put in during 24 hours continuous operation.

The machine was driven by a gasoline engine, mounted on the same truck. The gasoline drive for the mixer made it possible for the hoisting engineer to give a maximum of attention to the operation of hoisting, as the little gasoline engine required attention only at the time of changing crews, to see that fuel and oil were supplied in sufficient quantities for the succeeding shift. Hot water was used for the mixing operation from the beginning of the shell to the end of the job.

THE SLIP JOINT.

Figure 2 shows how the slip joint at the bottom of the wall was made. The graphite paste was spread first upon the concrete foundation and the sheet copper was laid immediately on top of this. Upon this copper sheet the wall was built

This view shows also the construction of the wooden circular ribs of the jacking forms, supported on pieces of the segmental combination wood and plaster forms, which were previously used to secure the smooth wall of the elevated portion of the center of the foundation. This wall or rise is shown clearly just below the wooden rings

PROGRESS OF FORM WORK

The illustrations, Figs. 3 and 4, show two stages in the erection of the jacking forms Fig. 3 is a view taken from the hoisting tower and Fig 4 is a detail of the assembly of the wooden circular ribs of the jacking forms with the vertical jack yoke posts and the radial truss covered by the working floor

The central radial truss was designed strong enough to support the dome forms after the shell had been poured. This eliminated at a stroke all necessity for a scaffolding or staging from the ground up. Figure 4 shows the floor laid over the radial truss. The floor boards were put down in such manner as to effect the least cutting of the lumber, as this decking was used again for the surface of the dome forms. Figure 4 also shows the completed forms shortly after they began to move. The jacks which accomplished the vertical movement of the forms are shown at each yoke. An exterior scaffold was attached to the movable forms for the use of the workmen who rubbed down the exterior surface with carborundum rubbing bricks, as called for by the specifications. As the height increased an outside railing was added to this hanging exterior scaffold. Figure 5 is a view of the dome forms.

The Fulton standpipe was designed by the Concrete Steel Engineering Co of New York City. Messrs. Chas. F. Bornefeld and E. H. Harder were resident engineers. One of them was present days and the other nights while the concrete shell was being poured. The A. Monsted Co. of Milwaukee was the contractor, being represented on the construction work by Mr. Allen D. Whipple.

Comment Upon the Value of a Detailed Map of the Distribution System of Small Water Works Installations.

To the Editors: It seems to me that there can not be too much stress laid upon the necessity of keeping complete records and detailed information about all the conditions that exist, or are known to exist, in the distribution system of a small water works plant. I say "of a small water works plant" because it is now the general practice to keep such records for large plants.

A good illustration of the value of these records came under my observation today. We had about 100 ft. of 6-in main to lower to conform to new street grade and before commencing the work I ordered examined the valves, etc., necessary to cut out this section of street, to see that they were in good working order so that in case an accident occurred while lowering the main we could be sure that we could cut off the water without delay. Upon examination of the maps which are supposed to show valves, etc., we found that there were no valves shown and so we had to hunt for them. This we did and the result of our hunt showed us that there were valves but such arrangements of valves and mains as I have never seen before. On one street the cross main was not connected, but was laid under the main we wished to lower. On the next cross street the main was connected, but there were no valves. On the next cross street the main was laid under the main we were lowering but there were no records of where the new mains and the old mains were connected. The mains that were on the cross

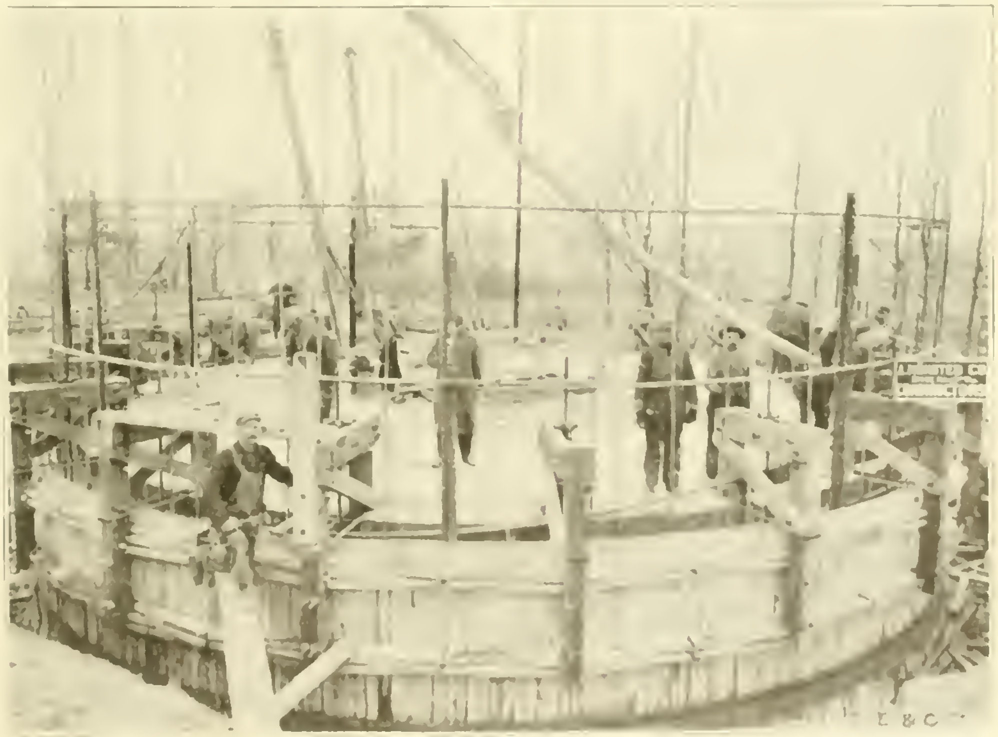


Fig. 4. View of Completed Forms Shortly After They Had Begun to Move. The Jacking Forms Are Also Shown.

streets were laid about ten years after the main we were working upon. It seems that to save a little money the consulting engineer had ordered every other connection left out, so that we had practically two interlocking but not gridironing systems to control, with about enough valves to control one system.

After spending one day in locating all the valves we went around to close them so that we would be sure they would cut off the water.

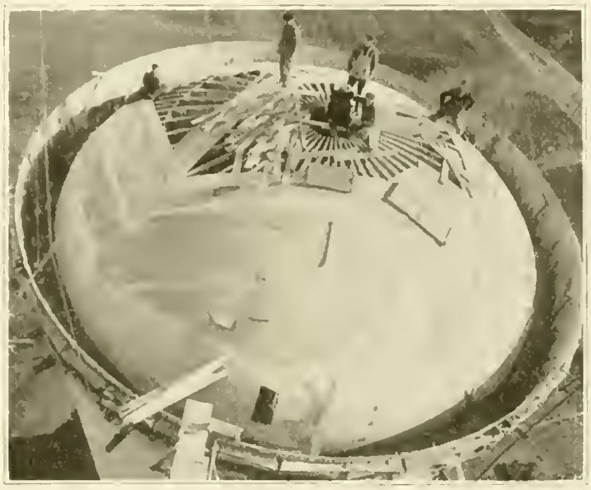


Fig. 5. View Showing Centering For Dome of Fulton, N. Y., Standpipe.

As we went to close one of them the top piece of the stem and the head nut were thrown up into the air and were immediately followed by a regular gusher. The main was under 60 lbs. pressure and there were no hindrances to the full and free flow of water through the stem hole in the top of the valve. After we succeeded in cutting off the water from this valve, we found that its stem had been broken some time ago. (At least two years ago, as the writer has had charge that long and the break occurred before he took charge.) The stem broke just above the collar which holds it stationary in the valve body, and the person who repaired it took out the gate and the lower section of the stem and reset the top section in the stuffing box with a full charge of packing and let it go. This arrangement held until we started to turn the stem, when it blew out as before stated.

Now the point of my argument is this. Had there been any records made on this valve and filed where they could have been found, we would have known that this was a dead valve, that the stem was just sticking in the body of the valve, and we would have repaired it long ago. As it was the only record was in the mind of the man who did the work, and as he is not here his mind record was not available for use when needed. For that reason we had to prospect and find the trouble, as we did unexpectedly and much to our discomfort

Had there been a proper map made of this system, such as I have tried to get made, we would have saved one-half the cost of it this forenoon and would still have had the map for future use.

Very truly yours,
M'Kean Maffitt.

Superintendent Florence Water Department
Florence, S. C., April 7, 1911.

(We publish this letter to present an example of the trouble which is sure to arise in any city which fails to keep proper records of the nature and location of its underground water works structures. Illustrations of this character could be multiplied almost indefinitely. The condition is well understood by engineers and in considerable measure is due to engineers, but an additional illustration of its possibilities for making trouble is worth while. However, the chief thing we would have our water works readers observe is their duty to urge upon city councils the need of adequate records of all underground structures. The need must be urged again and again until it is recognized and suitable action taken. The technical municipal employe must be patient in his efforts to educate non-technical officials; he must expect to have much resistance to overcome. Educational work of this character is an important function of the technical man in municipal service. No amount of impatience on his part, due to the slowness of city officials to appreciate the importance of certain matters advanced by him from time to time, can excuse him for failing to continue his educational efforts. These observations are general and are not directed at our correspondent, although suggested at this time by his letter. —Editors.)

Removal of Anchor Ice by Means of Compressed Air at Moline, Ill., Water Works.

Many and varied methods have been proposed for the removal of anchor ice from water supply intake structures. Many plants have methods of their own and these have met with varying degrees of success. The use of air for this purpose is not new. The success of this method at the Moline, Ill., water works, however, has been such that troubles formerly encountered there with anchor ice are no longer experienced, and the application of the method to Moline conditions is here described as of possible interest to some water department officials. Our information is taken from a paper by L. A. Fritze, chemist of the Moline water department, before the Illinois Water Supply Association.

The water is supplied to the plant through two intake pipes, a 20-in., 350 ft. in length, and an 18-in., 3,500 ft. in length. The water flows by gravity to a well and is then pumped to the filter plant. The intake pipe with this air attachment extends something like 350 ft. into the river. It is open and unprotected at its mouth; no crib or other device has been built about the pipe.

Formerly a centrifugal pump was so arranged that it could be connected to the 18-in. line, and in an emergency the water could be drawn through this line until the well was filled, and then the valve on the choked 20-in. line was opened, letting the full pressure bear against the obstruction. The depth of the well is about 20 ft., and the head usually obtained by this means is but 15 ft. This system, while helpful at times, could not be relied upon, for as long as the 18-in. line remained open things were satisfactory, but just as soon as the ice began to choke the plant was at a standstill.

During the past year the chief engineer, Mr. Buck, has attempted to improve the plan by connecting the blower used to air wash the filters to the 20-in. line. The blower used is of the Root make, giving a large volume of air under low pressure; that is, up to about 10 lbs. At times for very short periods the pressure has been forced up to 30 lbs. The connection between the blower and the line is through a 2½-in. pipe connected to the intake pipe, near the raw water well end.

On nights when the temperature conditions indicate a slight freezing, the height of the water in the raw water well is carefully watched. When the level begins to drop, the blower is started and in a very short time the line is completely cleared. At times when the line has been choked tight, the blower has without fail cleared the line in a few minutes. The maximum pressure needed or used for this purpose runs about 10 lbs. per square inch. From the accompanying diagram the layout of the raw water well and the air connection can be clearly understood.

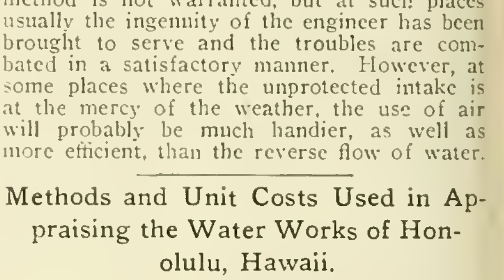
The Mississippi River water, turbid and colored, seems to be affected in no way in the formation of anchor ice, other than the temperature. It is claimed by some that the appearance of this ice occurs only on certain nights—nights following clear days, or a clear night, not cloudy ones, etc. Clear days and cloudy days alike seem to have little influence over the ice problem in the Mississippi River so far as we have been able to observe. The main trouble usually becomes noticeable after midnight and remains until the appearance of daybreak.

There are a number of plants where the local conditions are such that the use of this method is not warranted, but at such places usually the ingenuity of the engineer has been brought to serve and the troubles are combated in a satisfactory manner. However, at some places where the unprotected intake is at the mercy of the weather, the use of air will probably be much handier, as well as more efficient, than the reverse flow of water.

Methods and Unit Costs Used in Appraising the Water Works of Honolulu, Hawaii.

I.—DISTRIBUTION SYSTEM.

The latest annual report of Mr. J. W. Caldwell, Superintendent of Public Works of Honolulu, contains a very interesting and valuable report on the appraisal of the city's water works and sewerage systems. The appraisals were made for rate making purposes. Existing financial records were very faulty. The flat rates in vogue were unscientific and the empirical basis upon which they were predicated could not be determined by the present officials. The appraisals were made under the immediate direction of Mr. A. C. Wheeler, assistant superintendent of public works, from whose report to Mr. Caldwell the information here given has been taken. The organization of the appraisal staff is indicated in the accompanying diagram.



Plan Showing Intake Pipe and Compressed Air Connections in Raw Water Pit of Moline, Ill., Water Works.

The present article relates to the appraisal of the water works distribution system. Of special interest is the discussion on the life of water pipe of various materials. The reader will not fail to note the short life assumed for such pipe, and the local conditions affecting the life of pipe will also be noted. A second article will relate to the appraisal of the pumping stations and machinery, reservoirs and miscellaneous structures. The articles relate primarily to the methods and unit costs used in making the appraisals.

GENERAL APPRAISAL METHOD USED.

Records of costs of the Honolulu Water Works have not been as definite as might be wished. It is very difficult to determine exactly what the cost of pipe and properties were for work done under the monarchy. Again, original costs of buildings, pumps, and some pipe lines are difficult to obtain owing to the fact that the labor was contracted and the materials furnished by the territory and that complete records are not obtainable. In general, it has been assumed that the original costs are, in this case, misleading, indeterminate, and unsatisfactory.

The method used in making this appraisal was to compute by one way or another the cost of duplication, based on prices of labor and material for Jan. 1, 1914.

DETERMINING DEPRECIATION USED IN VALUING THE DISTRIBUTING SYSTEM.

In assuming the correct amount to be set aside as depreciation fund, the appraisers were confronted with the difficulty that the water works had never set aside anything for maintenance, repairs or duplication, at the end of the life of the factor. For instance, a pipe, the probable life of which is 35 years and which has been in the ground for 17½ years, may be assumed as having one-half the value of a pipe which is just installed. Its probable useful life is only 17½ years as compared with 35 years for a new pipe. Had the water works department set aside a fund for renewal of this pipe at the date of its installation, it would be comparatively easy to compute the required sums to set aside each year, which would replace the pipe at the end of its life. However, no sums of any kind have yet been placed at interest for duplication of any of the water works property.

The appraisers were therefore confronted with the problem of determining exactly what depreciation rate to assume for the different factors making up the total of the water works property. There are two means of solving this difficulty for pipe, which constitutes about one-half of the water works property.

First Solution.—All pipe lines can be assumed as having an equal length of life, which length must necessarily be less than the actual length of life of any pipe which will now be laid. For instance, if we assume 35 years as the probable life of pipe in the ground in Honolulu, for our deductions in this solution, we must assume, say, 15 or 20 years and work out a depreciation rate from this term of years and distribute it indiscriminately over all the pipe lines in the city. This rate would give a sinking fund large enough to replace the old pipe, but the old pipe renewed would be paid for out of sinking funds collected for the other pipe. For instance, pipe 15 years of age would be paid for out of the sinking fund for new pipe recently laid and the sinking fund for the new pipe would be built up from increments set aside, say, 15 years hence.

Second Solution.—Another method would be similar to this, but computed in a different manner. Let us say that each pipe must pay for itself at the end of its life; then the pipe which has been in the ground 15 years will have a sinking fund figured on its own value and such that it will amount to the value of the pipe at the end of 20 years. This rate would be 3.36 per cent. The pipe which has just been laid and whose length of life is 35 years would have a depreciation rate of 1.36 per cent. This method would absolutely guarantee sufficient money in the sinking fund to renew any and all pipe when the pipe is 35 years of age. While the depreciation rate can be computed for any one year at considerable expense, it would seem better to compute the total sinking fund by this method, then average the total value and total sinking fund, compute the proportion required and assume this as a proper rate for a period of say 5 or 10 years. This will hold true for a certain period, but gradually as the pipe becomes 30 years of age and is renewed and the new pipe takes its place with the longer term of years at this high rate, the sinking fund will be found excessive and must be reduced. Such a method as above described, however, allows a very easy and simple computation, and while it is evident that the amount will become more and more excessive, it is believed that with the system now suggested for reporting pipe lines and connections, the depreciation rates in future years can be re-computed without considerable trouble.

PIPE SYSTEM.

Pipe in Honolulu has been reported for the use of this appraisal on mimeograph sheets. These sheets were made out to show the pipe, valves, manholes and hydrants. The present worth, depreciation, and cost of duplication has been computed and set down on these cards and summaries made of the totals. The pipe constitutes almost one-half of the property of the water works department.

LIFE OF PIPE.

Cast Iron Pipe.—In order to work out the present worth and depreciation values, it is necessary to make an assumption as to the expected and probable life of pipe in the ground. This applies also to hydrants, valves, and manholes.

Some research was made in Honolulu to determine the probable life of pipe. Old pipe was taken up and examined and compared with new pipe of the same kind.

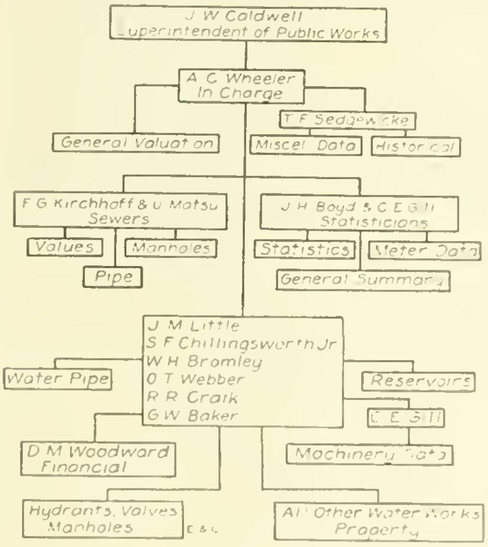


Chart Showing Organization for Appraisal of Water Supply and Sewerage Systems of Honolulu, Hawaii.

Test of Old Pipe.—On Oct. 1 a test was made on old cast iron water pipe at the Beretania St. pumping station. The pipe is known to be at least 36 years old, having been in use for 30 years and sustaining an average water pressure of 30 to 40 lbs. This pipe was taken up and laid on Hobron Ave., Kaimuki District, six years ago and has been there sustaining a pressure of from 60 to 70 lbs. This pipe in appearance seems to be sound and practically as good as new. Examination shows the metal to be clean at the fracture and sound throughout with very few pit holes and cracks.

The pipe was tested by hydraulic pressure from the inside. First, the ends of the pipe were faced off in a lathe, then plates were fastened at each end, held in place by a rod through the center of the pipe. Considerable difficulty was encountered in making the joints tight. Asbestos washers were first used with valve rubber seat. The rubber was found too soft and the asbestos disintegrated in the water so as to allow of leaks. The apparatus was then dismantled and re-assembled, using gasket rubber for washers and winding the inside rod with asbestos wicking. Arranged in this manner the pipe withstood a pressure of over 500 lbs. and then leaked slightly, due to the elongation of the inside rod. The rod used was 1 in.

The pipe being filled with water, the hydraulic pump was started and the pressure run up to somewhat over 500 lbs. The gage read only to 500 lbs., while the needle showed considerably beyond the 500-lb. mark. This pressure was the limit of the capacity of the pump and the pipe showed no signs of breaking. No cracks or leaks occurred.

An examination of the pipe made during and after the test showed no effects from the pressure and it may be assumed that the pipe is safely able to carry any pressure which would occur in ordinary use.

Example of 30-Year-Old Pipe.—It was found necessary to remove about 500 ft. of 2-in. cast iron pipe from Robello lane, owing to insufficient pressure. An examination of the pipe taken up shows the inside heavily incrusted. This incrustation does not diminish the area by more than 10 or 20 per cent, but is so rough and has so many protuberances and ridges that the friction of the water in the pipe must be very greatly increased. Most of the pipe appears solid and sound. The pipe, however, was pitted in places almost entirely through the shell. These pits are invariably on the outside and not on the inside. From the appearance of these pits, it is evident that the pipe could not have lasted much longer, even though no trouble was encountered on the inside from incrustation. This pipe was condemned by the superintendent of water works and sold for its metal value. Here is a case, then, of a cast iron pipe, whose length of life was not over 30 years, and whose usefulness towards the end of 30 years was much diminished by the incrustation.

Five men have been consulted as to their estimate of the probable life of cast iron pipe in the ground in Honolulu. Their estimates varied from 35 to 50, with an average of 42 years.

Mr. Sedgwick, chemist and statistician, has been instructed to obtain such data as he could regarding the length of life of pipe in Honolulu: He states as follows:

> It was the general opinion that the life of an ordinary pipe here is about 30 years. I have not found much proof of this.
>
> My own opinion is that the life of the pipe will depend on the soil in which it is laid, and the character of the water that passes through it. I believe some pipe would not last more than 25 years, and some would last 50 years, the pipes being of the same quality. The mountain waters will have a tendency to clog the pipe, so reducing its delivering capacity. The artesian waters that contain considerable quantities of salt will have a tendency to eat the pipe.
>
> In one instance a 4-in. pipe in use for 25 years was found so heavily incrusted that a 1-in. pipe could not be put through it.

During a conversation with Mr. Sedgwick he stated that it was his opinion that in making an appraisal we would be perfectly safe in assuming 30 years as the average length of life, while he felt sure of this, he also thought that we would not be far wrong in increasing this to 35 years.

Severe Local Conditions.—There are records of pipe in England and the United States which have been in the ground considerably longer than 30 years; steel pipe, wrought iron pipe, and cast iron pipe having been known to last 50, 75 and even over 100 years. Pipe taken up in London, which was laid in 1803, showed incrustation, but very little reduction in cross section of metal. It must be remembered, however, that Honolulu lies on the seashore, that some of the streets are sprinkled with salt water, that street car lines run near many of the pipes and that the water which is used in Honolulu will incrust pipe in 25 years to a large amount. If it is possible to clean the pipes it is our belief that 40 or 50 years or even more would be suitable life, but as this process is uncertain, and has never been used in Honolulu successfully, we have assumed that 35 years would be the proper length of life for pipe in Honolulu.

Life of Steel Pipe.—Fourteen-inch galvanized steel pipe in Maui has been known to last 12 years, while 20-in. pipe in other places has been almost useless at the end of 8 years. Sixteen-inch pipe laid in Honolulu in 1898 was condemned in 1906, although a portion of this line is used to the present day. Thirty-inch riveted iron pipe has been in use 30 years.

The Honolulu Gas Co. has reported that black iron pipe protected with red lead has been totally destroyed in three years in the soil of Honolulu. This may be unusual or it may be that the gas pipes are more subject to corrosion and electrolysis than water pipe. It has been assumed for the purpose of this appraisal that 10 years is a reasonable length of life for steel and galvanized steel pipe.

Wood Stave Pipe.—There is a very small quantity of wood stave pipe in Honolulu. A portion of this lies in the Upper Makiki Valley and the balance in Upper Nuuanu Valley. The pipe in Upper Makiki Valley was laid in 1907 and is now in very poor shape. The pipe in Nuuanu Valley was laid in 1906 and is now in fair condition.

TABLE I—HONOLULU PIPE HAULING DATA, 1911.

Size of Pipe.	Length. Feet.	Length. Miles.	Total Cost.	Cost Per Ton Mile.
Misc.	600	. . .	.95	.95
Misc.	8,300	1.57	1.50	.995
Misc.	6,500	1.23	1.50	1.200
Misc.	8,000	1.53	1.00	.653
Misc.	7,000	1.33	1.50	1.130
Misc.	23,000	4.36	2.50	.573
18 ins.	27,000	5.12	3.00	.575
12 ins.	6,000	1.14	1.00	.887
12 ins.	30,000	5.69	3.10	.545
8 ins.	7,000	1.33	2.00	1.505
8 ins.	6,000	1.14	1.74	1.530
6 ins.	20,000	3.78	3.00	.795
6 ins.	13,000	2.46	2.40	.975
6 ins.	15,000	2.84	2.00	.704
6 ins.	21,000	3.98	2.50	.627
Misc.	13,000	2.46	1.40	.570
Misc.	7,000	1.33	1.00	.754
Misc.	21,000	3.98	2.10	.527
Average				.864

THE VALUATION OF WATER PIPE.

Local Cost Records Not Suitable for Unit Prices.—The records of the water works department do not go into the details of cost of laying water pipe to such an extent that we would be warranted in assuming the figures as a basis for the cost of re-production. Such records as we have are usually composed of items for extras, and omitting items for hauling, superintendence and sometimes material, pig lead, yarn and cast iron pipe have been purchased in large quantities and sometimes delivered to some central point from which they were distributed to the different jobs. The cost of hauling to such a central point was uniformly omitted in the cost of separate and isolated jobs. Similarly, in most cases, the amount of pig lead or other materials used was omitted in the statement of total cost of the pipe.

Much of the work of laying water pipe in the last two or three years has been done by contract. The territory would contract for materials, such as valves, fittings, lead and yarn and supply these articles to another contractor. This contractor would then excavate the trench, sometimes haul the pipe and materials, and sometimes have the pipe and materials delivered to him; back-fill the trench and re-metal the surface. Extras were sometimes included in the way of taking up old pipe and hauling the same away, and lowering the pipe. Some contracts included the care and repair of the service connections.

TABLE II—COMPLETE ITEMIZED COST OF CAST IRON PIPE LINES IN SOFT MATERIAL IN HONOLULU.

Items in Dollars per Linear Foot.

Size of Pipe. Ins.	Laying.	Lead and Yarn.	Hauling.	Iron +5%.	Total.
2	$0.337	$0.0200	$0.0200	$0.263	$ 0.640
3	.337	.0251	.0200	.351	.7331
4	.337	.0314	.02560	.456	.8500
5	.337	.0388	.0333	.592	1.0011
6	.337	.0466	.0420	.746	1.1716
8	.337	.0654	.0615	1.092	1.5559
10	.415	.0827	.0840	1.491	2.0727
12	.500	.1021	.1095	1.942	2.6536
14	.593	.1226	.1380	2.455	3.3086
15		. . . . .	. . . . .	. . . . .	3.6500
16	.692	.1367	.1700	3.020	4.0197
18	.800	.1692	.2100	3.620	4.7992
20	.915	.1798	.2400	4.280	5.6148
24	1.168	.2113	.3250	5.780	7.4843
30	1.600	.2913	.4720	8.370	10.7333

It is evident, therefore, that it is rather complicated to work out the cost of reproduction from this data.

Time and Method of Obtaining Unit Costs.—It is, therefore, assumed that the proper way to arrive at the cost of reproduction would be to take each separate item of cost, compute it as closely as possible, and total the sum. Prices were obtained from firms for materials, hauling and of accessories for the pipe system. The cost of laying was computed from present and old records in the water works department, from data furnished by contractors and the gas company, and from a general knowledge of the cost of construction in Honolulu. All costs have been made, as regards time, for Jan. 1, 1914.

Unit Costs of Pipe.—The problem of appraising the pipe lines of the water works department has been divided and computed as follows:

The itemized cost of the pipe was computed with a unit cost for hydrants, valves, connections and meters. Connections have been ignored as a separate item and have been included in the regular length of pipe and in the extra number of joints figured for the given length of pipe. This was deemed as accurate as any other practical method.

CAST IRON PIPE LINES.

Unit Cost of Metal.—The weight of pipe has been computed for all sizes from 2-in. to 30-in. These figures were taken from "Gillette's Hand-Book of Cost Data" and may be assumed as accurate. These weights are, however, class "C" pipe, which has been used almost entirely in the water works department. The weight per pipe length in pounds was taken from the table, the weight per foot in tons was computed, allowing 12 ft. per pipe length. A ton, as a unit weight was used in preference to a pound in both hauling and cost of pipe. From quotations from the Honolulu Iron Works and previous contracts, it has been assumed that $40 per ton is a fair and accurate price of pipe, delivered in Honolulu. Five per cent breakage was then allowed for broken pipe, defective pipe and for pipe which would have to be cut in order to fit in the pipe line. With these data, the computations have been made for the cost of the pipe for each linear foot of pipe line.

Unit Costs of Lead and Yarn. The quantity of lead and yarn used in caulking joints has been taken from "Gillette's Hand-Book of Cost Data," checked from other sources and was first computed in weight for each joint. The cost of lead has been assumed at $110 per ton or 5.5 cts. per pound. Yarn has been assumed at $100 per ton or 5 cts. a pound. Recent figures and quotations show that these assumptions on the prices of lead and yarn are about correct. With these prices the cost for lead and yarn per joint was computed. While the length of pipe is uniformly 12 ft., joints have been assumed to occur every 10 ft. in order to allow for pipe cut off for any reason and for connections to valves, hydrants and fittings. The cost of lead and yarn was then reduced to a linear foot basis and assumed in the regular computation of cost of pipe.

TABLE IV—COMPARATIVE PIPE LINE COST DATA.

	Page 676, "Gillette's Cost Data," using Honolulu prices.	Appraiser's estimate.
6-in line, Ohio	1.156	1.1616
6-in line, Porterville, Cal., page 672, 300 ft. soft, 538 ft. hard	1.503	1.4216
8-in, same, 800 ft. soft, 1,758 ft. hard	1.364	1.7759
4-in, same, 900 ft. soft, 1,946 ft. hard	0.809	.850 (soft) 1.254 (hard)
10-in same, 138 ft.	2.085	2.0727 (soft) 2.5707 (hard)
4-in. at Alliance, O., page 671	.436	.850
6-in., not corrected for	.5951	1.1616
8-in labor	.7863	1.5559
10-in. and	1.0697	2.07
12-in. costs	1.3245	2.50
14-in., Wilkes-Barre (partly corrected)	2.970	4.0206

Unit Cost of Hauling.—As all pipe laid in Honolulu has been shipped into the country from the coast, the item of hauling plays a rather indefinite part in the cost of the pipe line. Pipe at Kaimuki may cost less for hauling than pipe on Punchbowl, although the distance to Kaimuki is perhaps three times as great from the wharf as is Punchbowl. Again the size of pipe would make considerable difference in the cost of ton-mile haul.

Pipe for different jobs have had in many cases a double haul, first from the wharf to the store-house and second from the store-house to the pipe line. Thus pipe a short distance from the wharf has in some occasions had a high cost for hauling, due to the fact that the pipe had to be stored before it could be installed. In looking over hauling data and referring to the contract of 1911 for hauling large amounts of pipe, we find that the cost of hauling pipe from the wharf or store-house to the site has run between 95 cts. and $3.10, as shown in Table I. Worked out, the cost of hauling amounted to 86.4 cts. per ton mile. Assuming a 2½-mile haul and allowing 10 per cent for fittings and breakage, would give the cost item for hauling as $2.365 per ton. Considering all things, it is believed that this item would approximate the cost of hauling for all parts of Honolulu. Quotations have been received from both the Honolulu Construction & Draying Co. and Hustace-Peck Co., stating that their estimate for hauling is $1 per ton mile for pipe to any and all parts of Honolulu.

TABLE III—COMPLETE ITEMIZED COST OF CAST IRON PIPE LINE IN HARD MATERIAL IN HONOLULU.

Items in Dollars per Linear Foot.

Size of Pipe. Ins.	Laying.	Lead and Yarn.	Hauling.	Iron +5%.	Total.
2	$0.741	$0.02	$0.02	$0.263	$ 1.044
3	.741	.0251	.0200	.351	1.1371
4	.741	.0314	.02560	.456	1.2540
5	.741	.0388	.0333	.592	1.4051
6	.741	.0466	.0420	.746	1.5756
8	.741	.0654	.0615	1.092	1.9599
10	.913	.0827	.0840	1.491	2.5707
12	1.1000	.1021	.1095	1.943	3.2546
14	1.3050	.1226	.1380	2.455	4.0206
15	. . . .	. . . .	. . . .	. . . .	4.4350
16	1.523	.1367	.1700	3.020	4.8497
18	1.760	.1692	.2100	3.620	5.7592
20	2.100	.1798	.2400	4.280	6.7998
24	2.662	.2113	.3250	5.780	8.8783
30	3.520	.2913	.4720	8.370	12.6533

Unit Cost of Laying.—Under this heading of laying has been included the cost of trenching, laying, caulking, yarning, and back-fill. Data on this item are very scarce in this office. During the years 1904 and 1905 the department of public works laid considerable pipe by day labor. Summaries of these reports changing the prices paid for labor, caulkers and foremen to the present prices, would give about 50 cts. a foot of 12-in. pipe for soft material and $1 a foot for hard materials, as indicated in Tables II and III, respectively. As some of the reports of this year have been made in a careful and detailed manner, we have assumed these data as being fairly correct. Unfortunately, however, we have no data for other sizes than 12 in. except one report on an 8-in. pipe. It seems fair to assume that the cost of laying pipe would vary with the size of the trench. Assuming for a 12-in. pipe, a trench of 18 ins. by 30 ins., or an area of 540 sq. ins., and computing the sizes of all trenches from 8 ins. to 30 ins., we have figured out the cost proportionately to the area and have checked these results with all possible available data. Table IV gives the comparative results of the cost of pipe laid in the states with the material costs of labor wages altered to suit the present conditions. While considerable discrepancies show in this table, it must be remembered that in most cases the specifications called for more work than the simple laying of pipe; work such as lowering of existing pipe, removal of existing pipe, difficult connections, service connections and other work has been required. In other cases, certain materials have been furnished which are not easily and accurately computed, so that it must be assumed that these data are not such that they should be used as a criteria for the cost of duplication of the water works system.

Unit Cost Summary.—Tables II and III show a summary of these four items with the total cost per foot. In the work these totals have been simplified by taking the nearest even figures to two decimals and applying them directly to the reports of pipe lines now existing in Honolulu. These totals are the unit values applied to the mimeographed reports on pipe lines, and the total value of the pipe was computed by multiplying the total length in feet by this unit value. It will be noted that the valves, manholes and hydrants have been computed separately and that their depreciation rate and values have been kept distinct from the pipe.

DEPRECIATION AND VALUE.

Manholes—The manholes in the water works vary greatly in size and condition. As a rule, they are made just large enough to accommodate the valve, and manhole frames are made as small as possible. While their costs vary considerably, it has been assumed that on an average, $20 apiece would replace the manholes. The length of life assumed for a manhole is 20 years. This assumption has been made considering the rather light construction used on some of the lines and the heavy traffic which passes over other lines.

Hydrants—The value of hydrants has been obtained from present prices, being $18.50. The life assumed is 30 years, which is rather small, if proper care is taken of the hydrants.

Valves.—The cost of valves has been obtained from quotations at the present time as shown in Table V. Valves have been assumed to last 30 years, the same as the hydrants.

Meters.—The value of meters has been assumed as $15. This includes the cost of installation. It has been known that in certain portions of Honolulu meters will wear out much faster than in other parts. Certain meters in the Nuuanu System have worn out in 18 months, due evidently to the grit carried in the water. It is generally assumed that meters will last 20 to 30 years, but for the purpose of this appraisal, 10 years has been assumed as the length of life.

Service Connections.—The value of service connections has been computed at an average of $5.25 apiece, and the probable length of life at 15 years.

TABLE V—VALUE OF GATE VALVES.

Size. Ins.	Value.
2	$ 5.05
3	7.50
4	10.40
5	13.50
6	16.70
8	27.00
10	45.00
12	62.50
14	78.00
15	104.00
16	130.00
18	175.00
20	212.00
24	200.00
30	550.00

Buildings

Cost Accounting on Construction Work—Detailed Description of an Efficient System.

II.

In our issue of April 8, 1914, we published Part I of a paper, by Mr. Leslie H. Allen of the Aberthaw Construction Co. of Boston, giving details of an efficient cost accounting system in use by this company on construction work. We are giving the remainder of this important paper in this issue.

Section IV.—The Work in the Office—Labor Records.

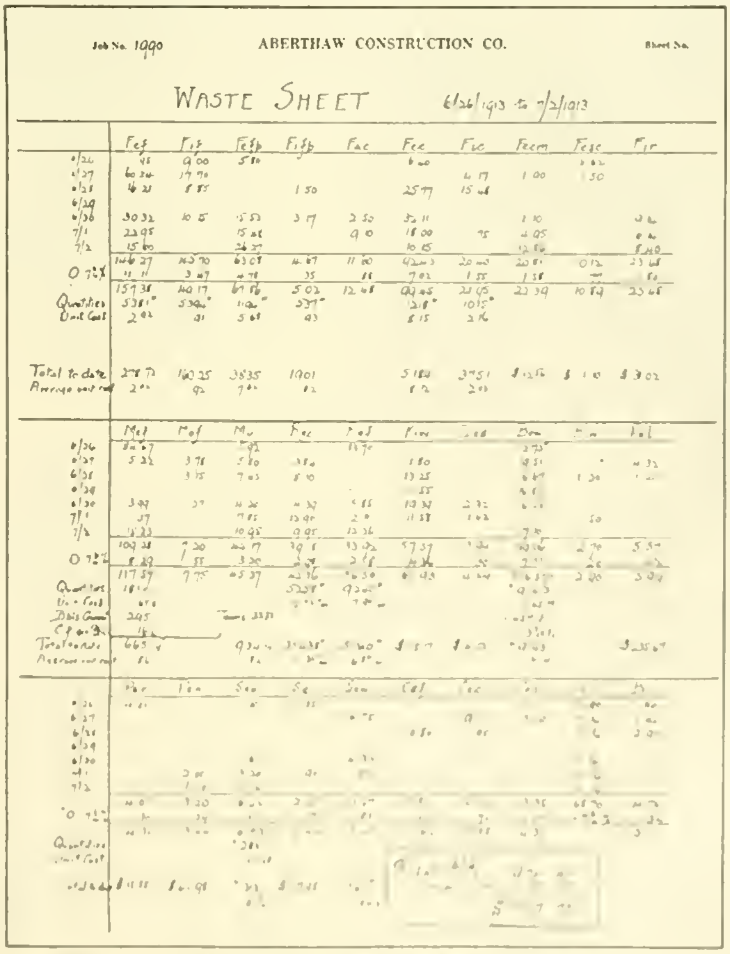
(A) WORKING UP THE TIME SHEET.

As soon as the time sheets come into the office, the total cost of each operation is worked out and entered in the column headed "For Office Use" on the time sheets, as shown in Fig. 6. At the end of the week the totals are drawn off on to an abstract sheet, shown in Fig. 10, and the items are totaled. The overhead labor expense (superintendent, timekeeper, waterboy, etc.) is then distributed by adding a percentage to each item, and the complete cost of each operation for the week is determined. The quantities are then written underneath each one and the unit costs worked out. They are then entered on the office record sheets, which will be described in the next section, and under them is entered the total quantity of work done to date (which is obtained from the office sheet) and the average cost of the work to date.

(B) THE WEEKLY SUMMARY.

Figure 11 shows a copy of the weekly summary of labor costs. The top line shows the total amount spent during the week on the items. The second line shows the quantity done, and the third line the unit cost. There are then left two or three lines for notes; in the case of concrete the number of barrels of cement used and the number of cubic feet of concrete obtained with a barrel of cement is noted. The two lower lines show the total quantity of that kind of work done to date and the average cost. Four copies of this sheet are made every week, and are furnished to the heads of the firm and the general superintendent, and one copy goes to the superintendent in the field. He also has a copy of the analysis. With these two he is able for himself to see how the costs of his work are running and how they compare with the estimate. Most of our superintendents also work out daily unit costs in the field on the larger items of the work, to keep more closely in touch with their costs. We believe in letting our men know just what we expect of them in the way of costs, of letting them know just how they are coming up to or bettering our expectations. Since we adopted the plan of letting them know what the weekly costs were there has been a marked improvement in the unit costs and a real enthusiasm for getting low costs.

I may be asked why we do not "go one better" and work out daily costs in the office instead of weekly. The reason this is not done is not on the ground of expense—although it would be a very expensive proceeding—but because if we worked out the cost of every item each day we should have such an overwhelming mass of figures to study that we should never have any clear idea as to how the job stood, and then, again, the costs would show such astonishing variations from day to day that in many cases they would be useless. For example, suppose a line of sheet piling set up and braced one night, but not driven right down until the next day, following our rule (to report quantity when work is done) one day would show a large expense in money and no quantity, the next a large quantity with small expense and exceedingly low unit cost.

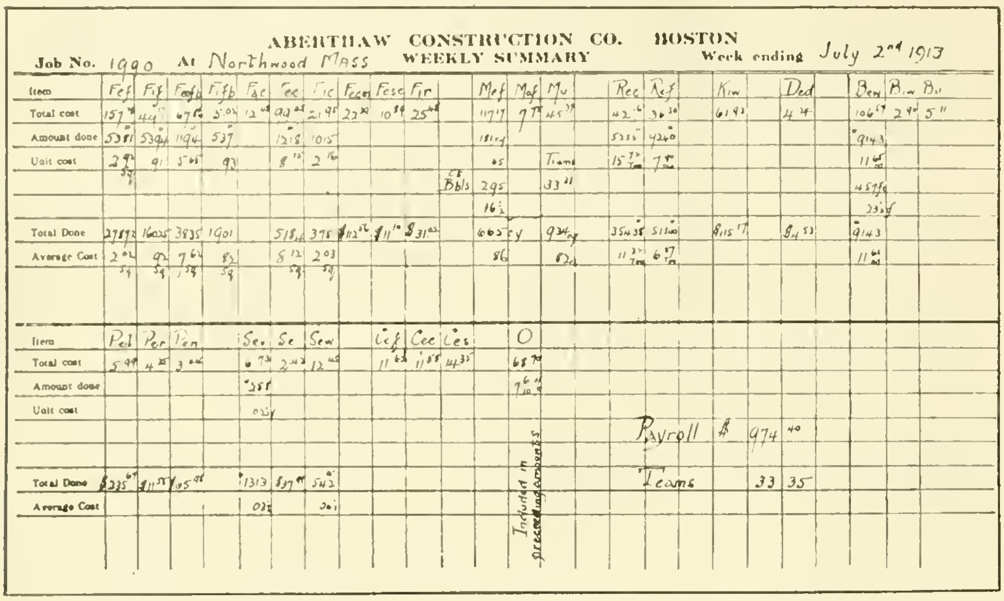

Job No. 1990 ABERTHAW CONSTRUCTION CO. Sheet No.

WASTE SHEET 6/26/1913 to 7/2/1913

Fig. 10. Waste Sheet—Quantities and Unit Costs.

(C) THE OFFICE LABOR SHEETS.

Figure 12 shows the regular record of the labor which is kept in the office. This sheet shows only a few items and there should be about ten or twelve sheets for every job. It will be seen that the first column contains the date, the next the quantity of work done during the week, the next the unit cost of that work, and the next column the total cost of that work. The next column contains the average cost of the work to date, combining the week's work with all the previous weeks. In the case of concrete, two additional columns are used, giving cement used and the proportion of the mix. These are entered up from week to week from the abstract mentioned previously. The totals are kept in pencil at the foot of the columns and altered from week to week, so that it is a simple matter to add one week's work to the total of the preceding week and work out the average unit cost.



ABERTHAW CONSTRUCTION CO. BOSTON
Job No. 1990 At Northwood Mass WEEKLY SUMMARY Week ending July 2nd 1913

Payroll $ 974.40
Teams 33.35

Fig. 11. Weekly Summary.

On some items, such as forms, the cost of making, erecting and stripping of forms is carried in separate columns, and another column is kept for the total unit cost of the work to date. This is obtained by adding the totals of the three money columns and dividing by the number of square feet erected. This inclusive unit cost does not give quite such an accurate idea of the cost as the three subdivisions do, as all the making is done at the beginning of the job and some of the stripping may not come until a good while after, but is in many cases a very useful figure to have, and it is, of course, the figure which compares with the analysis which is made out at the beginning of the job. As already mentioned, when the cost of the item for the week has been entered, the average is worked out and then the resulting average and the total quantity to date is transferred back to the waste sheet from which the weekly summary is made out.

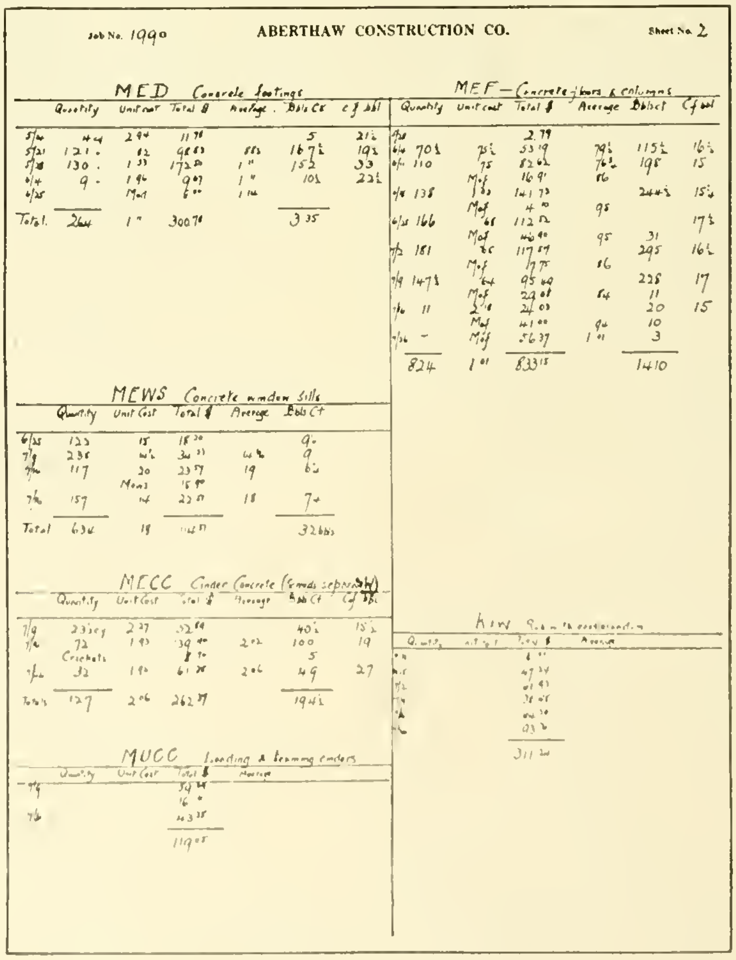
Section V.—In the Office—The Material Records.

Figures 13 and 14 show a part of the material records kept in the head office. These are entirely distinct from the bookkeepers' records and are not a ledger account. No merchants' names appear as a rule, but chiefly quantities of materials and costs.

The records are kept on loose sheets and at the start of the job columns are headed for each item that appears in the analysis. In some cases these items are subdivided. Then every item of expense is entered under its proper heading regularly as the work goes on. Freight in every case is entered with the item to which it relates. Demurrage also is entered in the columns of "Cement," "Brick," "Lumber," etc., as the case may be. The column headed "Cement" will also contain items of freight, credit on empty bags, tests, demurrage, etc., so that the final price per barrel of cement that appears in the final summary may be several cents higher than the price entered on the original order, especially if many empty bags have been lost. There is no column for all lumber. Lumber is entered under "Plant," "Forms," "Roof Plank," "Cofferdams," "Temporary Buildings," etc., according to the use it is to be put to. If lumber is bought for sheeting trenches and afterwards used for forms, it is first entered to the excavation item and then its second-hand value is credited and charged to forms. Credits are entered in red and in the same columns as debits, as there are very few of them and it would be cumbersome to have to keep double columns for each item to provide for possible credits.

To get hold of the information entered in these sheets the procedure is as follows: All bills are sent to the job to be checked and returned to the head office, from whence they are paid. As soon as the receipted bills come in they are sent to the cost accounting department. They are then entered up in a waste book and at once returned to the bookkeepers. One waste book only is kept, and the bills are entered as they come in, a page at a time being kept for each job number, and another page taken when one is filled. When the bills are checked at the job, the material clerk notes on same what the material was used for (as in the case of lumber just referred to). This information is usually put in by using the timekeeper's code and then there is no question in the cost accounting department as to where to charge any unusual item. The use of the waste book is simply to save time. The weekly labor summaries take precedence in the cost accounting room, and on the last three days of the week we are much too busy getting these out to give any time to the material sheets. These can be entered up later in spare moments. Once a month, at least, each job is brought up to date and the total compared with the bookkeeper's ledger.

All orders are also examined by the cost accounting department, and a note sheet kept on each job of all large orders and subcontracts, so that when a monthly statement is made of the financial standing of the job these can be included. The saving or loss on the estimate when a contract such as painting is sublet is not made when the final payment has been made, but when the order is given, and should be taken into account then. We do not, however, keep a record of any but the big orders, as the thousand and one small items of nails, bolts, tools, etc., are billed and paid for very soon after receipt and quickly find their place in our records from the bills.



Job No. 1990 ABERTHAW CONSTRUCTION CO. Sheet No. 2

MED Concrete footings
MEF—Concrete floors & columns
MEWS Concrete window sills
MECC Cinder Concrete

Fig. 12. Office Labor Sheet.

Section VI.—The Monthly Statement.

Figure 15 shows the statement which is prepared monthly to ascertain the amount of money saved or dropped on the job. These are not made for every job on the same day, but by taking two or three jobs in turn each week we make it part of our regular routine without undue pressure at any time.

The weekly summaries showed labor costs only. If any item (excavating, for instance) was costing 20 cts. per yard more than the estimate, it did not show how many dollars the total loss amounted to. Every job fluctuates from week to week. Some items are over the estimate and some show a saving. This statement brings these items all into view in such a way as to show how serious an over-run may be, and the final footings show within a very small amount just how the job stands.

The way the sheet is made up is—first to copy from the analysis the description of all the items and place the unit prices in the column provided on the left. Then from the labor and material records to enter in the "Actual Cost" columns quantities of work done and materials purchased, with their unit costs and total costs to date. Then to work out in the "Estimated Cost" column the cost of the quantities done at the estimated prices. Then to enter all sub-contracts made in the "Actual Cost" column and the corresponding "Estimated Cost," and finally to work out the totals saved or lost on each group of items.

A glance at the sheet (see Fig. 15) will show some considerable variations from the original estimate. It is not within the scope of this paper to discuss the actual costs on this work, but it will be well to point out what is shown on the sheet and to give a few explanations as to local conditions to make things clear.

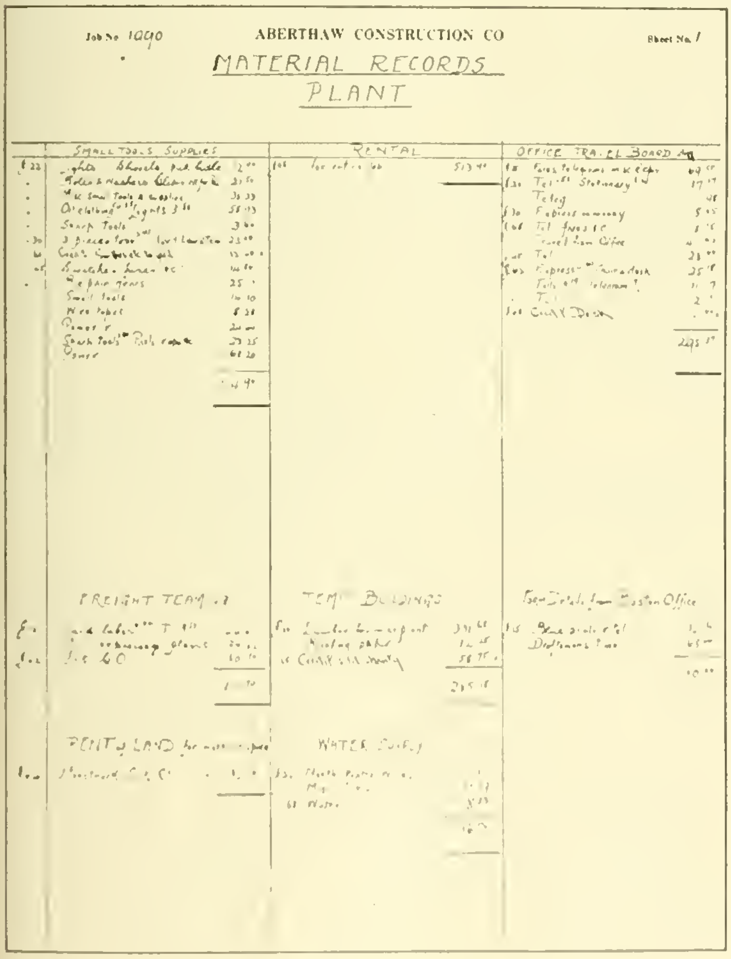

Job No 1090
ABERTHAW CONSTRUCTION CO
Sheet No 1
MATERIAL RECORDS
PLANT
SMALL TOOLS SUPPLIES
RENTAL
OFFICE TRAVEL BOARD
FREIGHT TEAM
TEMP BUILDINGS
WATER SUPPLY

Fig. 13. Material Records (Continued in Fig. 14).

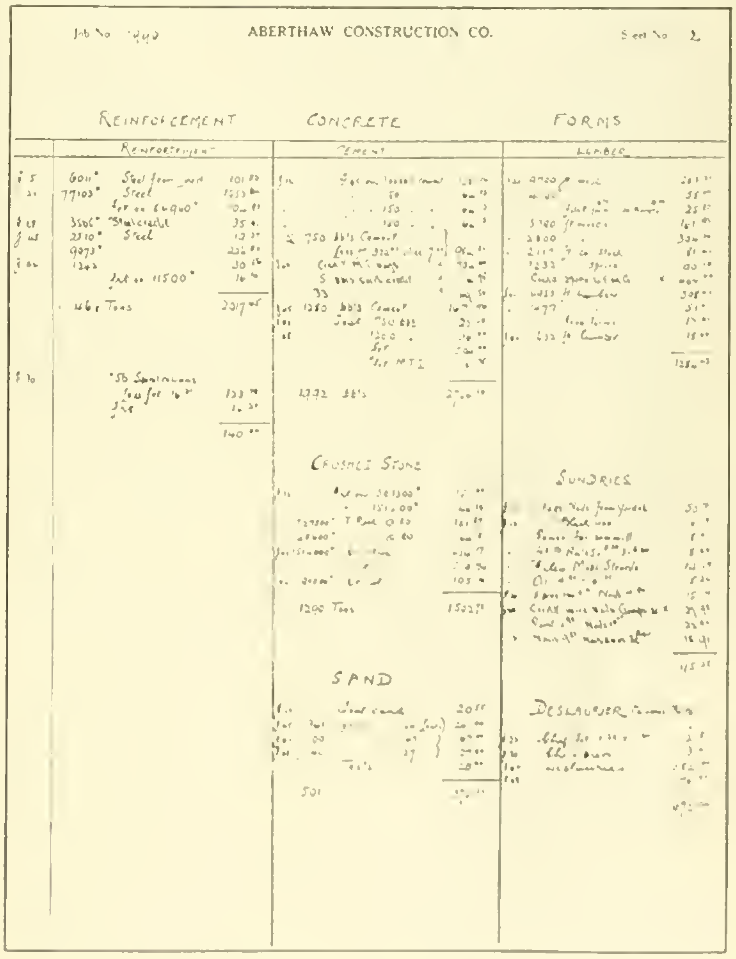

Job No 1090
ABERTHAW CONSTRUCTION CO.
Sheet No 2
REINFORCEMENT
CONCRETE
FORMS
CEMENT
LUMBER
CRUSHED STONE
SUNDRIES
SAND

Fig. 14. Material Records (Continued from Fig. 13).

It will be seen, first, that concrete labor is running very close to the estimate, although the quantity placed in the footings was 18 yds. in excess of that estimated. Plant shows a loss of $268, chiefly on the labor items. An old construction elevator was sent to the job, which was rather out of repair and several parts had to be refitted or remade before it could be erected. There is also a loss on the sand and stone. It was found that the crusher which had been counted on to supply us could not be depended on, and we therefore had to purchase from a quarry further away, which had a railroad delivery, and we unload and team from the railroad, which was about a mile away. As we were thus using teams regularly we decided to buy f. o. b. teams at the pit, instead of f. o. b. at the job, so as to keep teams steadily employed, so the statement shows a saving on sand and stone offset by an overrun on teaming. It is not necessary at this time to discuss the other variations, but simply to note that the difference between the savings and the losses shows a net loss of $230 on the estimated total.

The amount of contractor's profit does not appear on these sheets or in any of the cost accounting records, and the amounts of saving or loss in the monthly statement have to be added to or deducted from the estimated profit.

Section VII.—Monthly Comparison of Best Performances.

Table 2 shows a statement which we prepare monthly showing the unit labor costs of the three or four principal items of construction on all jobs during the preceding month. A copy of this sheet is sent to every one of our foremen and superintendents. This is not an essential part of our system, but it is awaited with great interest by our field superintendents and has proved a very useful factor in stimulating a general interest in the timekeeping and cost accounting work, and is an additional incentive to our men to try for low cost.

The costs shown on this statement are nearly always lower than our average costs, as they show the best work done month by month and do not bear any preliminary expense or other incidental items. They will give some idea of what can be done under favorable circumstances, but would not be a safe guide for estimating future work.

Section VIII.—The Final Comparison.

Figures 16 and 17 show the final comparison of estimated with actual costs. This is similar to the monthly statement (Section VI) and needs no further explanation as to the method adopted, but a brief notice of some of its chief features will be of interest.

The cost of laying wood [illegible] the top flooring was unexpectedly high, [illegible] as it was done in one week at the close of the job when the best men had been transferred elsewhere, and is a good illustration of the necessity of unceasing vigilance in superintendence to avoid sudden drops like this. The rest of the carpenter work was very well done, showing a saving of $914 on the forms. Plant continued to run high; 110 tons less of stone was used than estimated, owing to its being extremely well graded and therefore economical in use. Cinders cost three times the estimated amount, owing to there being none available at the owner's plant and no other factories near which could supply us. Some of the cinders had to be teamed four miles.

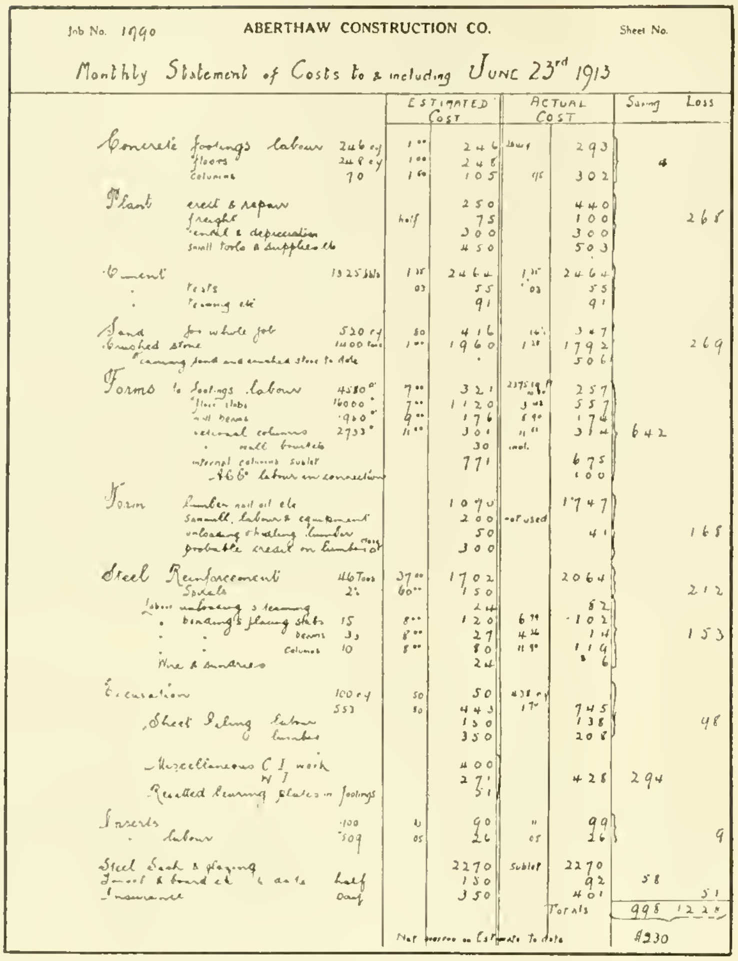
Section IX—The Final Summary.

When the job is completed and all accounts are paid, the figures are worked up into a final summary which shows the costs in the same manner as the original estimate. The cost of plant, cement, etc., is added to labor cost of forms, and in general figures are compiled which correspond with the prevailing methods of figuring construction work.

The final summary on the job from which these other exhibits have been taken is not yet complete, so we have taken one from another job, a storage building completed last year (see Figs. 18 and 19). It shows the method of setting out the figures so that, at a glance, any one of the important details of the cost can be referred to. The final summaries of all the jobs are then bound together in a loose leaf binder and are not filed with the job records. They are thus always at hand for ready reference when estimating future work.

Conclusions

The system has been used by the Aberthaw Construction Company for over two years, and it is interesting to look back and see what results have been attained by an accurate system of cost accounting. In comparing our estimates of two years ago with those made today, I find that I estimate concrete labor at least [illegible] per cent lower than then. On the other hand, I have found that not half enough money used to be carried for plant and tools.



Job No. 1990 ABERTHAW CONSTRUCTION CO. Sheet No.

Monthly Statement of Costs to & including June 23rd 1913

		Estimated Cost		Actual Cost		Saving	Loss
Concrete footings labour	246 c.y.	1.00	246	[illegible]	293		
floors	248 c.y.	1.00	248			4	
columns	70	1.50	105	1.15	302		
Plant erect & repair			250		440		
freight		half	75		100		268
rental & depreciation			300		300		
small tools & supplies etc			450		503		
Cement	1325 bbls	1.85	2464	1.85	2464		
tests		.03	55	.03	55		
teaming etc			91		91		
Sand for whole job	520 c.y.	.80	416	[illegible]	347		
Crushed stone	1400 tons	1.40	1960	1.28	1792		269
teaming sand and crushed stone to date					506		
Forms to footings labour	4580 □'	7.00	321	2375 sq. ft	257		
floor slabs	16000 □'	7.00	1120	3.48	557		
wall beams	1960 □'	9.00	176	8.90	174		
external columns	2733 □'	11.00	301	11.00	314	642	
wall brackets			30	incl.			
internal columns sublet			771		675		
466° labour in connection					100		
Form lumber nail oil etc			1070		1747		
Sawmill, labour & equipment			200	not used			
unloading & hauling lumber			50		41		168
probable credit on lumber [illegible]			300				
Steel Reinforcement	46 Tons	37.00	1702		2064		
Spirals	2½	60.00	150				212
Labour unloading & teaming			24		82		
bending & placing slabs	15	8.00	120	6.79	102		
beams	3½	8.00	27	4.24	14		153
columns	10	8.00	80	11.90	119		
Wire & sundries			24		6		
Excavation	100 c.y.	.50	50	438 c.y.			
	553	.80	443	1.70	745		
Sheet Piling labour			150		138		48
lumber			350		208		
Miscellaneous C.I. work			400				
W.I.			271		428	294	
Riveted bearing plates in footings			51				
Inserts	100	.10	90	"	99		
labour	509	.05	26	.05	26		9
Steel Sash & glazing			2270	Sublet	2270		
Travel & board etc [illegible]	half		150		92	58	
Insurance	day		350		401		51
				Totals		998	1228
		Net [illegible] on Estimate to date				$230	

Fig. 15. Monthly Statement.

TABLE 11. — COPY OF SAMPLE SHEET SHOWING MONTHLY COMPARISON OF BEST PERFORMANCES.

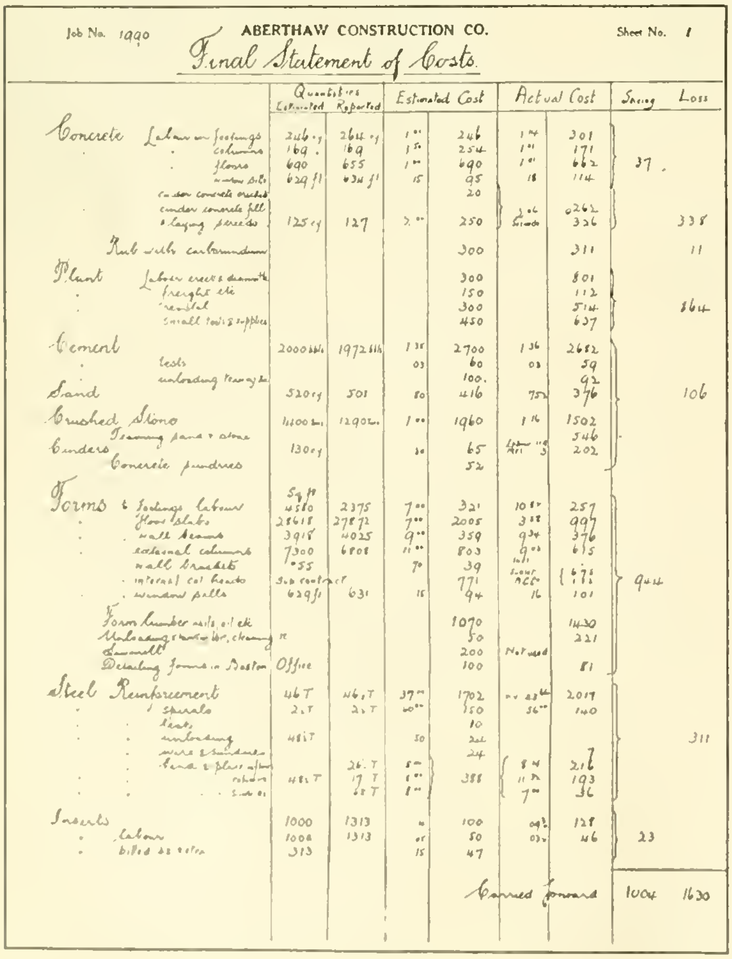
Principal Unit Costs for Month of October, 1911.

Concrete—Unload, mix and place.

N.B. 2 mixers on Job 941.

	Job No.	Cu. yds.	Unit cost.
Largest 6 days' run of concrete	917	313	$0.71
	941	2,205	.38½
	944	602	.62½
	946	558	.57
Largest 1 day run of concrete	917	107½	.80½
	941	451	.38
	944	150	.54½
	946	233	.51½
Lowest day's unit cost—floors	917	96	.68½
	941	408	.31⅓
	944	16½	.31½
	946	132	.38
Month's average unit cost	917	700	1.17
	941	5,030	.59½
	944	1,418	1.02
	946	393	.84
Forms—Erecting only.		Sq. ft.	Per 100 sq. ft.
Floor forms	917	45,628	$3.53
	941	291,016	3.92
	944	82,107	6.24
	946	24,568	7.10
Wall forms	917	None	None
	941	31,556	5.80
	944	3,855	9.59
	946	15,247	7.47
Column forms	917	3,725	3.17
	941	41,260	7.15
	944	11,494	9.08
	946	7,095	10.05
		Tons.	Per ton.
Placing floor steel (beam and slab)	917	82.18	$5.36
	941	886.5	4.17
	944	63.82	6.47
	946	37.00	6.85
Bend and place column steel	917	6.56	5.30
	941	14.33	2.84
	944	15.92	8.62
	946	7.33	14.71

Our labor costs on forms have come down over 25 per cent, hut I think that most of the saving on this item is due to improved designs and methods of erection than to a study of the unit costs. Steel reinforcement is handled for probably 10 per cent less than before. Our superintendents have all got a good knowledge of costs, and are really interested in following them from week to week. If a special and unusual piece of con-

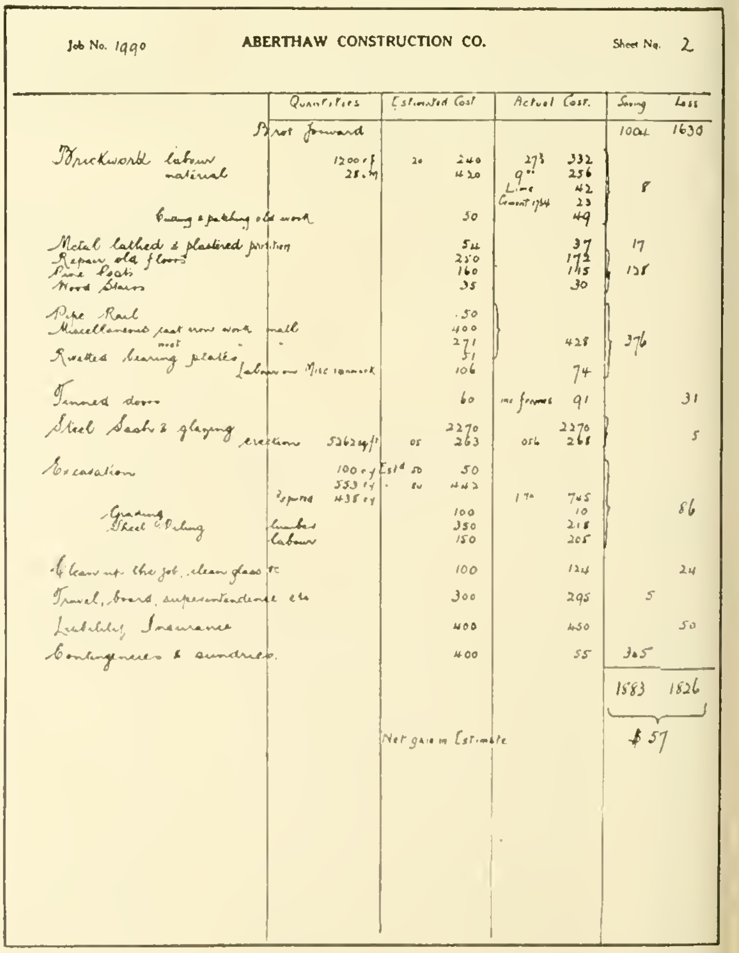


Job No. 1990 ABERTHAW CONSTRUCTION CO. Sheet No. 1

Final Statement of Costs.

	Quantities Estimated	Quantities Reported	Estimated Cost		Actual Cost		Saving	Loss
Concrete Labour on footings	246 c.y.	264 c.y.	1.00	246	1.14	301		
columns	169	169	1.50	254	1.01	171		
floors	690	655	1.00	690	1.01	662	37	
window sills	629 ft	634 ft	.15	95	.18	114		
cinder concrete [illegible]				20				
cinder concrete fill					2.06	262		
& laying screeds	125 c.y.	127	2.00	250	Screeds	326		338
Rub with carborundum				300		311		11
Plant labor erect & dismantle				300		801		
freight etc				150		112		
rental				300		514		864
Small tools & supplies				450		637		
Cement	2000 bbls	1972 bbls	1.35	2700	1.36	2682		
tests			.03	60	.03	59		
unloading teaming etc				100.		92		
Sand	520 c.y.	501	.80	416	.75	376		106
Crushed Stone	1400 tons	1290 tons	1.40	1960	1.16	1502		
Teaming sand & stone						546		
Cinders	130 c.y.		.50	65	[illegible]	202		
Concrete sundries				52				
Forms to footings labour	Sq ft 4580	2375	7.00	321	10.8	257		
floor slabs	28618	27872	7.00	2005	3.58	997		
wall beams	3918	4025	9.00	359	9.34	376		
external columns	7300	6808	11.00	803	9.03	615		
wall brackets	55		.70	39	incl.			
internal col heads	sub contract			771	[illegible]	671	944	
window sills	629 ft	631	.15	94	.16	101		
Form lumber nails, oil etc				1070		1430		
Unloading & hauling lbr, cleaning etc				50		221		
Sawmill				200	Not used			
Detailing forms in Boston Office				100		81		
Steel Reinforcement	46 T	46.1 T	37.00	1702	[illegible]	2019		
spirals	2.1 T	2.1 T	60.00	150	56.00	140		
tests				10				
unloading	48.1 T		.50	24				311
wire & sundries				24				
bend & place slabs		26.1 T	5.00		8.4	216		
columns	48.1 T	17 T	1.00	388	11.?	193		
[illegible]		5.1 T	1.00		7.00	36		
Inserts	1000	1313	.10	100	.09½	128		
labour	1000	1313	.05	50	.03½	46	23	
billed as extra	313		.15	47				
					Carried forward		1004	1630

Fig. 16. Final Statement of Costs (Continued in Fig. 17).



Job No. 1990 ABERTHAW CONSTRUCTION CO. Sheet No. 2

	Quantities	Estimated Cost		Actual Cost		Saving	Loss
Brot forward						1004	1630
Brickwork labour	1200 c.f.	.20	240	27½	332		
material	28 M		420	9.00	256		
				Lime	42	8	
				Cement 1.54	23		
Cutting & patching old work			50		49		
Metal lathed & plastered partition			54		37	17	
Repair old floors			250		172		
Pine posts			160		145	138	
Wood stairs			35		30		
Pipe Rail			50				
Miscellaneous cast iron work			400				
wrought			271		428	376	
Riveted bearing plates			51				
Labour on Misc ironwork			106		74		
Tinned doors			60	inc frames	91		31
Steel Sash & glazing			2270		2270		
erection	5262 sq ft	.05	263	.051	268		5
Excavation	100 c.y. Est'd	.50	50				
	553 c.y.	.80	443				
	reported 438 c.y.			1.70	745		
Grading			100		10		86
Sheet Piling lumber			350		218		
labour			150		205		
Clean up the job, clean glass etc			100		124		24
Travel, board, superintendence etc			300		295	5	
Liability Insurance			400		450		50
Contingencies & sundries			400		55	345	
						1583	1826
		Net gain on Estimate				$57	

Fig. 17. Final Statement of Costs (Continued from Fig. 16).

struction work is to be built, our men are keen to find out just what it costs.

Some time ago a large job began to show high unit costs; six weeks after the footings were complete the monthly statement showed that labor had overrun the estimate by $10,000. From any point of view but the cost accounting the job looked all right—well organized, and a large force of men all working busily. Each week's report showed unit costs as high as its predecessor. At the expiration of the six weeks the organization of the job was entirely changed; a new carpenter foreman was put on and several other alterations in the force were made, and at once costs began to go down. At the end of the whole job the whole of the $10,000 over-run had been picked up and a saving of about $700 made on the estimated labor costs. It is not often that such extensive changes are needed, but often on a job some item runs too high for two or three weeks. Special attention is given to that item until it is reduced to its normal level. I believe that our system is a reliable barometer, showing from week to week what our jobs are doing. The contractor does not want a post-mortem which, however interesting, does not bring back lost profits. He wants to know as the job goes along whether he is making money or not, where his profits or losses are, and whether his losses can be stopped.

Our system is elastic enough to take care of any special situations or furnish any information required. A little time back, I wished to analyze closely the cost of our form work with a view to furnishing our chief engineer with data which would guide him in making the most economical form designs possible. It was a simple matter for me in laying out the code for three jobs to subdivide the form work symbols by adding fourth letters, and get the cost and quantities of posts, joists, mud sills, panels and beam sides, etc., all reported separately and their unit costs worked out, so that we were able to compile data showing how much each post, joist, etc., cost to erect.

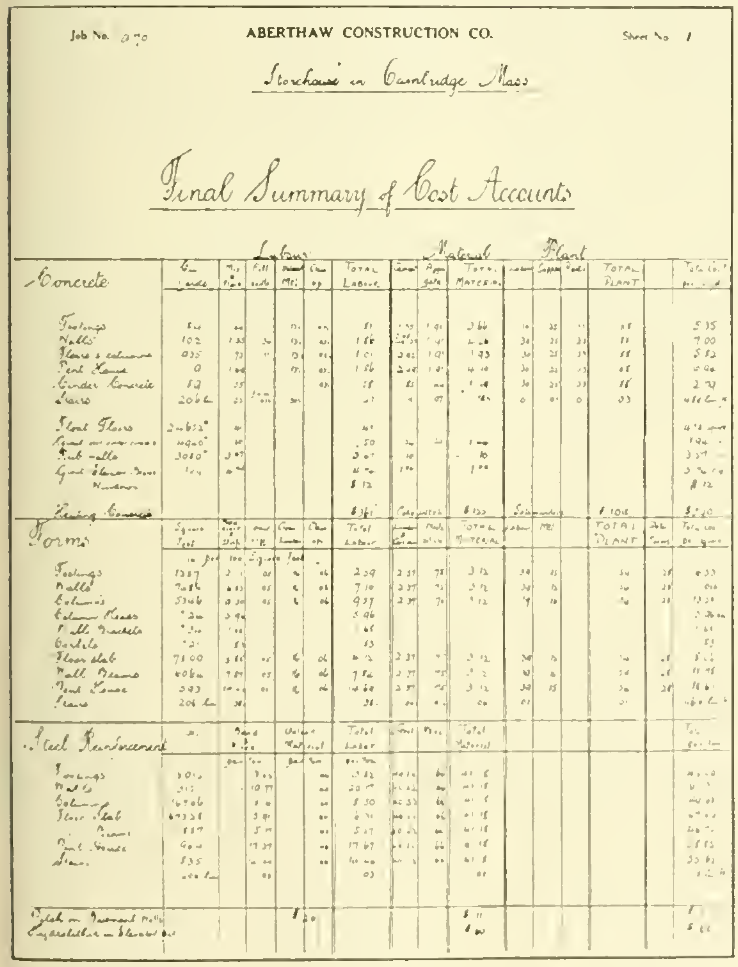
Although the system may at first sight seem to be complicated and costly, it is not so in fact. The work in our office is all done by my two assistants, who work with adding machines, and I give not more than one-fifth of my own time to supervising and directing it. Our total payrolls in the summer-time sometimes amount to as much as $18,000 a week, and this is all handled by these two men without difficulty, and leaves my chief assistant time to visit the jobs occasionally. In the field we spend little more than other contractors do on timekeeping. On a job having a payroll of $1,500 to $2,000 a week, one timekeeper at $15 would give his whole time to timekeeping. All the materials would be looked after by a material clerk at about the same wage. Larger and smaller jobs would have different organizations as their needs required.

I think the difficulties which are hardest to overcome and which need the most careful attention are: first, getting the timekeepers interested enough to study their work and divide the time rightly; second, seeing that quantities are reported for every item and reported correctly; and, third, having the figures handled in the office by men that understand them. My assistants have been picked not from the ranks of bookkeepers but from our own timekeepers. I am sure all the time that they know the meaning of the figures they are handling and can detect errors as they occur.

As I am sure to be asked why I have not mentioned the subject of curves or shown any in this paper, I will anticipate the question by saying that I do not draw curves of any of my cost figures. Where there is a progression in figures or costs between a known low point and known intermediate and high points, a curve may be drawn to find out without mathematical computation the intermediate figures. A great deal of labor may be saved in this way. But, in dealing with contractor's cost, the costs have to be computed first and the plotting of a curve of costs does not save any labor or show any new facts which were not easily discernible by the computed figures. All you get is a wavy line which may be satisfactory to the eye, but is of no real use and adds nothing to your previous knowledge. It does, however, give a visual impression which is helpful to some who do not readily take in the significance of figures. In my own work, if I were to draw curves I should have to keep over 200 up to date by alterations each week and should need a special assistant for the purpose.

For the establishment and conduct of a proper cost accounting system on construction work, the chief requisites do not seem to be so much a high degree of technical skill and expert knowledge as much as a common-sense view of the problem to be tackled and patient persistence in working out the details, refusing to be halted by any obstacles. It is easy to pay large fees to experts to install a system. It is not so easy to plug away week after week working out the system, patiently instructing men, detecting errors, correcting them, overcoming opposition, and *getting results*.

This paper has not been easy to write, and certainly has been very difficult to make interesting. As far as I know, it is the first time that a paper on the principles and methods of cost accounting on construction work has been presented, although many have been read on the subject of costs. If this effort accomplishes anything towards the establishment of accurate and uniform methods of figuring costs in contracting work, I feel sure that my company will not be the loser, and that I shall be more than recompensed for the time put in in compiling these remarks.


Job No. 270

ABERTHAW CONSTRUCTION CO.

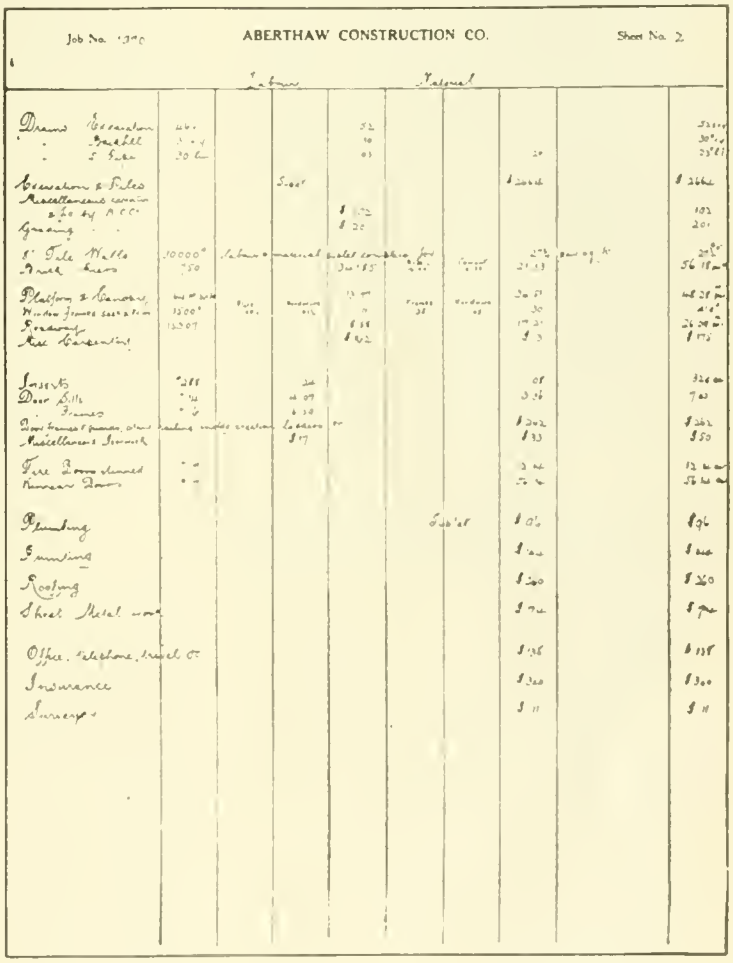
Sheet No. 1

Storehouse in Cambridge Mass

Final Summary of Cost Accounts

Labor
Material
Plant
Concrete
Forms
Steel Reinforcement

Fig. 18. Final Summary of Cost Accounts (Continued in Fig 19).


Job No. 270

ABERTHAW CONSTRUCTION CO.

Sheet No. 2

Labor
Material
Plumbing
Roofing
Sheet Metal
Insurance

Fig. 19. Final Summary of Cost Accounts (Continued from Fig. 18).

The Strength of Equipment for Handling Loads.

The engineer and the contractor are frequently required to pass upon the strength of various parts of the contractor's equipment. Accidents due to weak and faulty details are not uncommon. If such accidents are largely to be avoided it will be necessary for those in charge to become more familiar with the strength of handling equipment, instead of depending upon the manufacturer for all designs. The following data on the strength of equipment for handling loads were abstracted from a paper by Mr. Edward Godfrey, structural engineer, R. W. Hunt & Co., presented before the Engineers' Society of Western Pennsylvania. The working stress recommended by the author is smaller than that commonly used, but when we consider the fact that handling equipment is often used by men who do not have a clear conception of the strengths of various materials, low working stresses are advisable.

STRENGTH OF HOOKS.

Some time ago the writer was asked to pass upon the safety of a piece of equipment which was purchased as a 12-ton derrick. The first defect he found was a 6-ton hook. Other parts, although not so weak, were far below their requirements. It is not difficult to determine the safe load for a hook of any given shape and dimensions. The critical section is a horizontal one just beside the opening of the hook. In any of the hooks shown in Fig. 1 the bending movement at this critical section is *Pa*. If in the case of a plain round rod, curved into a hook, we assume that the inner diameter of the hook is twice the diameter of the rod the bending moment is $3/2Pd$. Using 10,000 lbs. per square inch as the safe fiber stress and $0.982\ d^3$ as the section modulus of a circular section, we have, when direct tension is considered,

$$P = 600\ d^2.$$

From this equation it is found that a ½-in. round rod of ordinary structural steel made into a hook of this proportion would sustain safely a suddenly applied load of about 150 lbs. A 1-in. rod would safely carry a load of 600 lbs.; a 2-in. rod, 2,400 lbs.; and a 3-in. rod, 5,400 lbs.

If the rod is flattened at the side of the hook the same amount of metal will give a stiffer and more efficient hook. Assuming that the rod is flattened at the side of the hook in the shape of an ellipse, with the long diameter equal to twice the short diameter and the inner diameter of the hook equal to the long diameter of the ellipse, we obtain the following:

The diameters of the ellipse of area, having an area equal to that of a circular section of diameter *d* are 1.414 *d* and 0.707 *d*. The section modulus of the same ellipse = $0.139\ d^3$, and the bending moment $Pa = 1.414\ Pd$. Adding the direct tension to the extreme fiber stress for bending we have, for a total fiber stress of 10,000 lbs.,

$$P = 870\ d^2$$

Hence a 1-in. rod made into a hook of these proportions will safely carry a load of 870 lbs.; a 2-in. rod, 3,480 lbs., and a 3-in. rod, 7,830 lbs.

The foregoing values are given to show approximately the capacity of hooks made of round rods and the advantage of the simple expedient of flattening the rods at the side of the hooks. For any special hook made of a round rod the capacity can be determined by using the actual diameter of the inside of the hook and of the rod used.

Standard hooks are made with the dimensions and proportions shown in Fig. 1 (c). The thickening of the cross-section near the inner part of the hook has the double advantage of bringing the center of gravity of the section closer to the line of application of the load and of increasing the area of metal resisting tension. A hook is more apt to fail due to the tension in the extreme fiber at the section of maximum bending than due to compression, and a tension failure is more disastrous, as the hook would not bend but would break suddenly without warning. Hence the importance of added tensile strength.

The style of hook shown in Fig. 1c was designed and its proportions worked out by Mr. Henry R. Towne, and is described in his treatise on "Cranes." The writer has taken these dimensions and has worked out the section modulus of the hook as well as the location of the center of gravity of the section. The section modulus for tension was found to be 0.15 times the cube of the diameter of the rod used in making the hook. A few of the stated capacities of these hooks are given in the accompanying table:

Diameter of rod, in inches.	Capacity of hook, in tons.
⅝	⅛
¾	½
1¼	1½
1⅜	2
2¼	5
3¼	10

The writer has tried a number of these hooks, and he finds that the extreme fiber stress for these capacities is close to 18,000 lbs. per square inch. This means that these hooks, in order to be safe for suddenly applied loads, should be made of high carbon steel and not of the ordinary grade of structural steel.

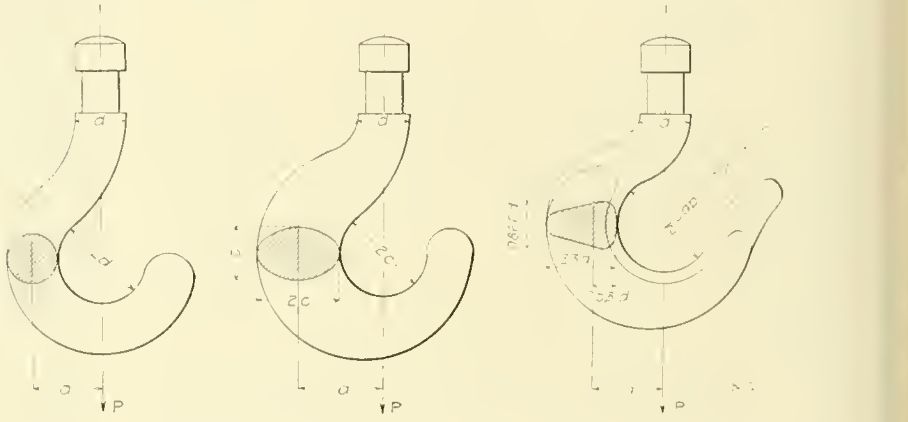


Fig. 1. Various Types of Hooks Used for Handling Loads.

Some special hooks are in use, such as those used for picking up girders by their flanges. If we assume that the lines *OA* and *OB* (Fig. 2) make angles of 60° with the vertical the force acting along each line is equal to the load lifted, or *P*. The critical section is at the bend, and the bending moment is *Pa*. Assuming that *a* is twice the thickness of the bar of which the hook is made we find that at 10,000 lbs. per square inch, when direct tension is considered,

$$P = 770\ bd.$$

By using this formula it is found that a 2x½-in. bar made into a hook of these proportions would safely lift a load of 770 lbs.; a 4x1-in. bar would safely lift a load of 3,080 lbs.; etc.

As in the case of the other hooks, the proportions may not be the same as those assumed here. In any given case these dimensions may be measured and the capacity of the hooks worked out by the method given.

The unit stress assumed in the foregoing is based on the use of ordinary structural steel. This unit stress takes into account the shock of a suddenly applied load. If high carbon steel were used the unit stress could be higher and the capacity of the hooks would be correspondingly higher.

High carbon steel is often used for hooks because a smaller hook may be used with the stronger material. If the fiber stress never exceeded the elastic limit of the material, high carbon steel would be entirely safe for hooks. The load on hooks, however, is apt to be quite variable and to be dropped some distance in the handling. If the fiber stress equals the elastic limit a sufficient number of applications would break the hooks. In softer steels the hook would probably be bent and thus give warning, but in high carbon steel it would snap off without warning. This is the danger of using hard, brittle steels where the loads are apt to overstress the steel. Soft steels are not free, however, from this danger of sudden and sharp breaks when subjected to repeated excessive stresses.

Hooks are sometimes annealed after they have been in service for some time to restore the metal to its original condition. The common notion is that steel crystallizes in service and that annealing restores the structure which is injured by this crystallization. This is one of the half-truths that give rise to much misapprehension. Steel can be made to give service indefinitely without any deterioration; furthermore, it is impossible to change the crystalline structure of cold steel. Steel will remain permanently uninjured if the fiber stress never equals or exceeds the elastic limit. Repeated application of fiber stresses close to or in excess of the elastic limit will eventually rupture the steel. Unfortunately, equipment for handling loads is very often made of such size that the loads handled, when shock is considered, are sufficient to stress the steel close to the elastic limit. The result is that such parts as hooks and chains are apt to fail in service, and that every precaution is needed to avoid such a failure.

Steel which has been badly treated by applying stresses close to the elastic limit is rendered brittle. Annealing restores, to some extent at least, the toughness of the steel and will often forestall failure.

The idea that steel is rendered crystalline by shock is a mistaken one. Steel is crystalline under any condition. If it is broken suddenly it will show the crystals more clearly in the fracture, whereas if the same piece is pulled apart slowly in a testing machine the crystals will be drawn out like fibers. Accidental breaks are almost always sudden ones—they show crystalline fractures—hence the common idea that steel crystallizes in service.

STRENGTH OF RINGS.

Rings are required on the ends of chains to throw over hooks and to pass the chain through in forming slings. Closed rings can be made of a much smaller diameter rod than open ones, but round rings must be of larger rods than the links of chain, which are usually of oval shape. Furthermore, since the rings are of larger diameter than the links of the chain they require a further increase in the size of the rod for equal safety.

If a circular ring whose diameter is *D*, made of a round rod of diameter *d*, has a load *P* applied in the manner in which it would be used in a chain the bending amount at the point of application of the load is

$$M = 0.1592\ P\ D;$$

and at the side of the ring,

$$M = 0.0908\ P\ D.$$

There is a direct pull at the side of the ring, the unit stress of which will be added to that found by this last bending moment, but in a ring of ordinary proportions this will not be sufficient to make the extreme fiber stress as great as that at the point of application of the load. Hence the greater moment is the criterion. Equating this to the section modulus of the rod ($0.0982\ d^3$) and using an extreme fiber stress of 10,000 lbs. per square inch the safe capacity of the ring is

$$P = \frac{6170\, d^3}{D}.$$

Hence a ring of ½-in. diameter steel, 3 ins. in diameter, has a safe capacity of 260 lbs.; a 4-in. ring of ¾-in. steel has a capacity of 650 lbs.; a 6-in. ring of 1-in steel, a capacity of 1,030 lbs.; a 7-in. ring of 1¼-in. steel, a capacity of 1,720 lbs.; and an 8-in. ring of 1½-in. steel a capacity of 2,600 lbs.

These values are very low, and for quiescent loads they could be twice this amount or more.

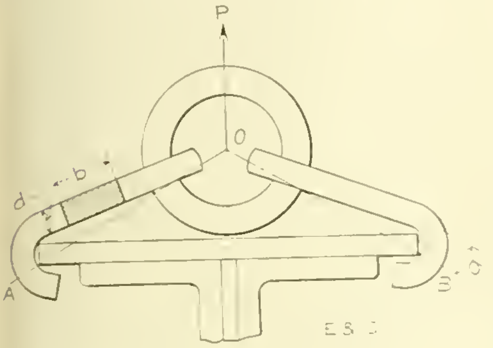


Fig. 2. Special Hook for Handling Girders.

Oval links are stronger than round ones because the bending moment is less. Without going into the exact theory to determine the bending moment on an oval ring it is close enough to consider that the strength of an oval ring is equal to that of a round ring of a diameter equal to the short diameter of the oval ring.

The writer knows of no standard in use for the safe strength of rings, so that he cannot make any definite statement as to the degree of safety usually employed. He has seen rings break in service, which is a good sign that they were overtaxed in that service.

STRENGTH OF CHAINS.

The safe loads which chains may carry are usually taken as one-third of the ultimate strength of the chain under test. The test load is usually one-half of the ultimate strength of the chain. This load is usually applied to the chain after its manufacture. It is by no means a safe load for the chain, although it is sometimes considered as such. Applied to the chain once, it elongates the links somewhat, and its application will usually detect any badly welded links; but repeated applications of this load would in time break the chain, as it stresses the fibers beyond the elastic limit.

When a chain is stressed to the point of rupture in a testing machine the links elongate and become straight and parallel along the sides. The failure is then one of direct tension on the steel, and the bending stresses of the original link are not present. For this reason the ultimate strength of a steel or wrought-iron chain is not in itself a proper criterion of its safe load.

There is another way in which a chain can be broken. This is by the process of repeated applications of a load which will stress the extreme fibers of the metal beyond the elastic limit. The links need not be appreciably distorted by this load, but the metal is gradually made brittle and will eventually fail at a load equal to or less than that which it has repeatedly carried. The break will be short, showing a crystalline fracture, and the majority of observers will say that the steel was crystallized in service, or that it was a poor grade of metal, or was burnt—all of which are wrong. This false interpretation of failures which result from overloading the equipment is so common that it has become engineering second nature to incorporate it into practically every report on such failures. Rings, hooks, bends, and chains all fall in the same class. Men reason that the member is perfectly safe because the load is carried once, or a number of times, or because the ultimate static load required to rupture the member is three or four times that which it is carrying. When failure takes place they say "bad steel or crystallization." This error of judgment has taken hold of the engineering profession, and from the writer's experience in trying to root out sundry other errors he believes it bids fair to abide for many years.

The standard safe loads used on chains in one bridge shop are shown in the following table:

Diameter of chain, inches.	Capacity of chain, tons.
⅜	1
½	2
⅝	3
¾	5
⅞	6
1	8
1 1/16	10
1⅛	11
1 3/16	12
1¼	14

These values are in agreement with common practice. The loads are about one-third the ultimate strengths of the chains.

By applying the formula, for round rings previously given, i. e., $P = 6,170 \frac{d^3}{D}$, and assuming the short diameter of a link to be 2½ times that of the rod ($D = 2\frac{1}{2}\, d$) we find that

$$P = 2,500\, d^2.$$

Solving this equation for various values of d we find that the safe capacity of a ⅜-in. chain, according to the above assumption is 350 lbs.; that of a ½-in. chain is 620 lbs.; that of a ¾-in. chain is 1,400 lbs.; that of a 1-in. chain is 2,500 lbs.; and that of a 1¼-in. chain is 3,900 lbs. It will be seen that these values are exceedingly low, something like one-seventh of the commonly used "safe" loads. Even if doubled, i. e., if 20,000 lbs. is taken as the extreme fiber stress, the values are still very low. There is one allowance which might be made in determining this theoretic strength of a chain, namely, a reduction in the lever arm because of the fact that the links fit together to some extent and do not have bearing only at a point. After making such allowance, however, it is plainly seen that the commonly used safe loads on chain are such as to stress the fibers to the elastic limit. This is why chains get brittle and break—why they need frequent annealing. It also explains why so many coroner juries have deliberated over a little piece of "defective," "burnt," or "crystallized" steel.

A stud-link chain is stronger than an ordinary chain because the studs prevent the links from flattening and thus decrease the bending moment. The advantage of stud-link chain over ordinary chain is much greater than that shown in ultimate tests because of the diminished bending moment, something that does not show up in ultimate test loads, but does appear in a large number of applications of a supposedly safe load.

Another point which should be considered is that a chain which is safe for lifting a certain load may be overstressed in lifting that load by improper hitching. Referring to Fig. 3, if the spread of the chain is such as to give an angle of more than 60° with the vertical, the stress in the two parts of the chain will be greater than the load lifted. It is not difficult to determine, in a practical manner, when the angle is too flat. If the parts of the chain are not equal to twice the vertical height *BD* the force triangle is flatter than an equilateral triangle, and the stress on the two parts of the chain is greater than the load lifted. A load of the capacity of the chain should not then be lifted.

A chain is usually made either of wrought iron or of soft steel that is capable of being welded. A wrought iron chain is better than a steel chain of ordinary quality, because it will better withstand the weather, and also because wrought iron welds are more reliable than steel welds. Furthermore, wrought-iron links are less apt to break abruptly; they will bend and give warning of approaching failure.

There is not much difference in the strength of a wrought iron and a soft steel chain. The grade of steel used in the manufacture of the latter has an ultimate strength of but little more than 50,000 lbs. per square inch, which is about the upper limit of wrought iron.

Chains, as well as rings and hooks, are commonly annealed to restore the toughness of the steel or iron. This annealing usually consists merely of building a fire and of throwing the chain into it; of heating it red hot, and then allowing it to cool. The toughness of the metal seems to be restored by this process, for a chain which is quite brittle can be rendered useful by annealing. The fact that the chain is brittle and needs to be annealed indicates that it has been subjected to service which has stressed the fibers close to or above the elastic limit. This common practice of annealing chains, as well as the frequent breaks which occur, are the outcome of the still more common practice of overloading equipment for handling loads.

STRENGTH OF CABLES AND ROPES.

Hemp and sisal ropes and steel cables are much used in handling loads. As a rule there is more safety in the use of cables than in the use of chains or hooks. When loaded excessively the cables will stretch appreciably and thus give evidence of their distress. They will not last long, however, especially with the small sheaves that it is customary to use in hoisting equipment. They seldom break suddenly and the wear or deterioration is plainly visible on the surface in broken wires, which is a decided advantage.

The ultimate strength of a few sizes of Manila rope are given in the following table:

Diameter of rope, inches.	Ultimate strength of rope, pounds.
¼	780
½	2,250
¾	5,000
1	9,000
1½	22,500
2	39,000

Sisal rope is about three-quarters as strong as Manila. The strength of ropes is quite variable because of the different qualities of materials used in their manufacture. The factor of safety should be five or six for a good life of rope.

The ultimate strengths of a few sizes of steel cables made of 200,000-lb. steel are given below, these breaking loads being for cables having hemp centers, with 6 strands of 19 wires each of so-called crucible steel:

Diameter of cable, inches.	Ultimate strength of cable, pounds.
¼	4,800
⅜	10,000
½	17,600
¾	38,800
1	68,000
1½	144,000
2	248,000

The writer is informed that there is very little steel cable made in this country of crucible cast steel. The material generally used is an imported open-hearth steel.

The proper safe load on a steel cable is governed somewhat by the use to which it is to be put. If it is to sustain a steady pull in a stationary position a factor of safety of three will be sufficient. If it is to be passed continually over small sheaves and subjected to varying loads the factor of safety should be five or six. In general the larger the factor of safety used the greater the life of the cable.

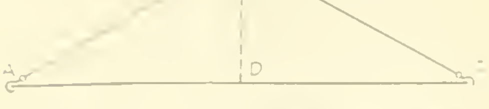


Fig. 3. Improper Spread of Chains

It is difficult to make an end hitch on a steel cable which will be as strong as the cable itself. The usual method is to [illegible] the ends of the wires, pickle them, and then put this prepared end in a conical socket and pour melted zinc around it. It might be reasoned that, as this is the strongest end detail, it is the proper one to use on the cable in practice. This is not the case, however, and a better end detail is made by looping the cable around a thimble and bolting with several clamps the free end to the cable itself.

This kind of hitch probably would not show such good results in a testing machine, because at the ultimate load of a cable it is drawn down in diameter and would slip through the clamps. At the working load, however, this would not take place. The socket detail is uncertain. It is impossible to tell whether or not the wires are held firmly in the metal without pulling the joint apart and thus destroying it. Furthermore, there may be unequal stresses on the several wires, as well as some tendency to anneal due to contact with the hot metal. It can be readily seen whether or not the clamped type of end hitch is properly made.

As will be more fully explained under "Sheaves," the sheaves and drums for cables should not be too small. Curving the cable to a short radius while under stress is detrimental. The outside wires are thereby overstressed, and the life of the cable is shortened. This is particularly detrimental when the drums or sheaves are close and are so located as to bend the cable in different directions; hence it is important in the proper use of cables to avoid such conditions in the design of the equipment.

Cables should be lubricated, not only to prevent rust, but to allow the wires to slide on each other so as to diminish the stresses due to the bending of the cables. Sometimes a mixture of linseed oil and pine tar is used as a lubricant; sometimes mica axle grease is used.

Cables in service gradually wear out due to the breaking, one after another, of the wires. These breaks are on the outside of the cable where they can be seen and counted. If they are quite generally distributed along the cable the reduction in its strength is not great, unless the breaks are quite numerous. Mr. S. Diescher states that when it is found that within the length of the pitch of a strand about 40 per cent of the wires are broken it is time to discard the cable. There are 114 wires in the ordinary cable, therefore when 46 of these are broken in the length of the cable in which a strand makes a complete whirl it is time to throw away the cable. This count can readily be made. It should be made at several points along the cable and the condition of the cable should be judged by the worst section, not by the best or the average.

THE SHEAVES

It is a general rule that the sheaves for ropes and cables should be of a diameter equal to 30 or 40 times that of the cable or ropes. Although this rule is observed in stationary sheaves, for elevators, inclines, etc., it is very generally ignored in equipment for handling loads. This is not a very serious matter, however, as it simply means that the cables will wear out in a shorter time. The rule can scarcely be adhered to in ordinary equipment, as 30-in. sheaves on a derrick using a ¾-in. cable would be very much in the way.

In the matter of the sizes of pins in sheaves there is room for improvement in common practice. If the ordinary methods of designing bridge pins were applied to sheave pins many of them would be found to be very highly overstressed. Many designers figure such pins for shear only. This is a mistake, as the bending moment is the governing factor in a pin, in fact it is the only one which needs to be considered.

The writer has checked the designs of many derricks and has talked with the designers. There is a general disregard of engineering principles, particularly in the matter of bending moments on pins, exhibited both by the designers and by their work. Pins 1¼ to 2.0 ins. in diameter are the common standards. The extreme fiber stresses computed in the usual way are often in excess of the ultimate strength of the steel.

When the pins are in iron or steel bearings they may be considered as having high bearing pressure, thus shortening the lever arm for bending, but when they are in wood the high pressure would crush the fibers of the wood. It is important in such cases that the pins be of extra large size so that they will not bend.

DERRICKS.

In addition to the parts already mentioned, all of which are used in derricks, there are other parts of derricks which need attention from the standpoint of strength of design, such as the mast, the boom, the guys, the stiff-legs, the goose-necks, the gudgeon pin, etc.

In a guyed derrick the mast must be high enough to allow the boom to pass under the guys. In many stiff-leg derricks the boom is twice as long as the mast. Derricks with masts 30 to 35 ft. long and booms 60 to 70 ft. long are not uncommon; such derricks need special care on account of the long boom, and the problems entailed thereby. One of these problems is the difficulty of obtaining support for the long and slender boom. Frequently the dead end of the boom line is hitched to the boom at about the middle of the boom in order to support the weight. The writer sees no objection to this so long as the stress is not too great for the size of the boom.

Hog rods with one or two intermediate posts are very useful in reducing the unsupported length of a boom. These should be used where slender booms are employed. It is best to use them on all four sides of the boom.

In designing a long boom the ordinary column formula is not applicable. The straight-line formula for columns, although excellent for bridge members, fails when an attempt is made to use it for slender columns. The Gordon-Rankine formula, with the constant usually given, is unreliable and unsafe when used for slender columns. The Euler formula, when properly interpreted, i. e., when it is understood that the formula gives the absolute ultimate load which a column will sustain, is the proper formula for slender columns. The common idea that the Euler formula gives the load which a slender column can hold in equilibrium is totally false and gives rise to error.

If the modulus of elasticity of wood is taken at 1,500,000, the unit stress in a column at its ultimate load is given by the formula,

$$K = 1{,}200{,}000 \frac{d^2}{l^2},$$

where K is the stress per square inch, and d and l are the minimum width and the length, respectively, of the boom, both in inches.

For example, a 11x14-in. boom 70 ft. long, supported for its own weight, will have an ultimate strength of 330 lbs. per square inch. For safe load, the factor of safety should be at least two. This boom would then be safe for 165 lbs. per square inch of load or for a total load of 32,300 lbs. This boom, if supported at its ends only in a horizontal position, will have an extreme fiber stress from its own weight of nearly 900 lbs. per square inch, yet it will not fail from this stress. It will, however, be very greatly weakened as a compression member. This boom is one which failed in a derrick while lifting a load of 18,000 lbs. The computed unit stress from this load was 170 lbs. per square inch, which is just about the allowable safe load, but the boom was not supported for its own weight, and in its inclined position the extreme fiber stress due to its weight was 590 lbs. per square inch.

The strength of slender compression members under lateral loads is something which has not received proper attention in engineering literature. The writer will not attempt to go into this further at this time except to point out that long booms should be so supported that their weight will not be such a large factor in producing stress.

The base detail of a derrick is important, especially in a derrick having a long boom, on account of the large horizontal force acting at the foot of the mast. The writer knows of one case where a derrick was set on two 12x12-in. timbers. With a low boom the horizontal force was sufficient to rock these heavy timbers. The sheaves for the boom line and the hoisting line were placed between these timbers. The solution in this case was to bolt the timbers together, using spacing blocks.

The gudgeon pin is the vertical pin at the top of the mast. The goose-necks, which are large iron straps fastened on the ends of the stiff-legs, pass over this gudgeon pin. This pin is sometimes too small and is often not firmly held at the extreme end of the mast, and as a result the fibers of the mast are crushed or the pin is bent. There should not be merely a ferrule driven in the top of the mast, with the pin in a hole in the wood, but there should be a steel plate over the end of the mast in which the pin fits snugly. Furthermore, this plate should be bent down over the sides of the mast, or a detail equivalent to this should be provided. The splitting of the mast should also be prevented by placing bolts through it near the end of the mast.

CRANES.

The various manufacturers of traveling cranes have standard cranes for given capacities. Data furnished by these companies show the clearances necessary in a building, the wheel spacing, the loads on the wheels, and the required size of rail. Roughly, the load on a pair of wheels is about double the capacity of the crane. The wheel bases usually run from 7 to 12 ft. A number of these standard tables are shown in the writer's book of "Tables."

The maximum moment on a crane girder will occur when the center of the span bisects the distance between the load near the center and the center of gravity of the system of loads on the span. With a single pair of wheels the maximum moment is given when one wheel is placed a distance from the center of span equal to one-quarter of the distance between the wheels, except when the distance between wheels is grater than 0.59 of the span when one load at the center of the span will give the maximum moment. With four wheels all on the span, the maximum moment is obtained when one of two inner wheels is placed a distance from the center of span equal to one-fourth of the distance between the two inner wheels.

Traveling cranes or gantries are of many forms and present many problems as to strength, one of the very greatest being that of securing adequate bracing. More wrecks have been caused by the lack of bracing in the false work and erection equipment than from any other single cause, if reinforced concrete wrecks be excluded. Timber travelers are usually well braced, but it seems that the larger travelers are relatively less efficiently braced than the smaller ones.

Roads and Streets

Practical Instructions to State Aid Road Foremen in Wisconsin.

Successful road construction is based upon careful and conscientious attention to small details. Methods and details of construction of course differ widely in different localities. Mr. A. R. Hirst, state highway engineer of Wisconsin, in Bulletin No. 4, Wisconsin State Highway Commission, entitled, "Instructions to County Highway Commissioners and Foremen for Building State Aid Roads in 1914," points out the methods which produce the best results in his state. The instructions are useful and are published in abstract as follows:

GRADING.

Staking Out.—Before starting to move any earth, stake out the new center line for the distance expected to be constructed that year, according to the center measurements given on the grade sheet. If the stakes do not line up, or if the new center line does not strike the old grade to the best advantage, set the stakes where they should be and make corrections on the grade sheet accordingly. This precaution will sometimes save costly errors in grading. It is best to set the grade and slope stakes before grading is begun. Make the cuts and fills indicated on the plan and grade sheet, filling slightly in excess to allow for settlement.

If the cut or fill given for any station on the grade sheet does not make the grade line work out in accordance with the way it looks on the plan or is obviously wrong, fix that station so it looks right and reasonable and build accordingly. About 500 measurements and computations are necessary before the grade sheet of a mile of road is completed, and it need not be surprising if an occasional mistake is made.

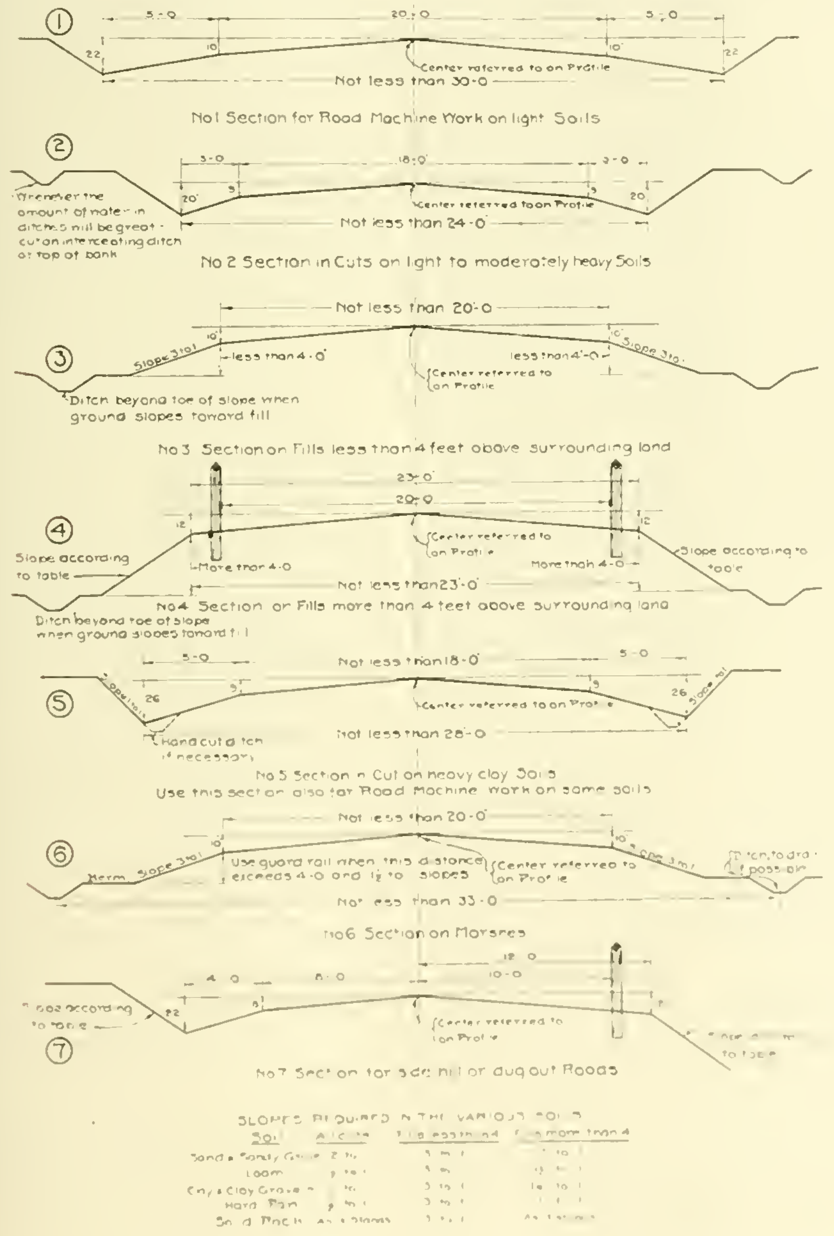

①
5-0
20-0
5-0
22
10'
10'
22
Center referred to on Profile
Not less than 30-0
No 1 Section for Road Machine Work on light Soils
②
3-0
18-0
3-0
Whenever the amount of water in ditches will be great - cut an intercepting ditch at top of bank
20'
3'
3'
20'
Center referred to on Profile
Not less than 24-0
No 2 Section in Cuts on light to moderately heavy Soils
③
Not less than 20-0
Slope 3 to 1
less than 4-0
Center referred to on Profile
less than 4-0
Slope 3 to 1
Ditch beyond toe of slope when ground slopes toward fill
No 3 Section on Fills less than 4 feet above surrounding land
④
23-0
20-0
12
12
Slope according to table
More than 4-0
Center referred to on Profile
More than 4-0
Slope according to table
Not less than 23-0
No 4 Section on Fills more than 4 feet above surrounding land
Ditch beyond toe of slope when ground slopes toward fill
⑤
5-0
Not less than 18-0
5-0
26
3'
3'
26
Center referred to on Profile
Hand cut ditch if necessary
Not less than 28-0
No 5 Section in Cut on heavy clay Soils
Use this section also for Road Machine work on some soils
⑥
Not less than 20-0
Berm
Slope 3 to 1
Use guard rail when this distance exceeds 4-0 and 1½ to 1 slopes
Center referred to on Profile
Slope 3 to 1
Ditch to drain if possible
Not less than 33-0
No 6 Section on Marshes
⑦
4 0
6-0
12 0
10-0
Slope according to table
22
3'
Center referred to on Profile
Slope according to table
No 7 Section for side hill or dug out Roads
SLOPES REQUIRED IN THE VARIOUS SOILS
Soil
Sand & Sandy Gravel
Loam
Clay & Clay Gravel
Hard Pan
Solid Rock
As shown

Fig. 1. Standard Cross Section for Roads in Wisconsin.

If a cut is producing too much fill, raise the grade equally at every point in cut and on fill until it balances. If not enough dirt is produced, lower the line equally. The Commission will not object to any change in line or grade that lessens work and secures essentially the same results that the plans sent out would have secured. If possible notify the Commission before making changes, but if not possible without delaying the work, adjust matters then and there and go ahead.

Slip scrapers will be found to be the best tool to move dirt distances up to 75 ft., wheeled scrapers from 75 to 300 ft., wagons (preferably dump wagons) for distances over 300 ft.

In those few cases where no surveys have been made, stake out a center line for the road and cut the humps and fill the hollows until you believe a satisfactory job is secured. Then shape up to cross-section with road machine. Do not crown the center portion of roads which are to be immediately surfaced.

Cross-Sections and Slopes.—No subject in road construction has more difficulties than the subject of cross-sections. Any number of cross-sections may be drawn and still some conditions will not be covered. The commission has for 1914 drawn up a number of cross-sections giving dimensions in cuts and fills for all conditions of soil and locality commonly encountered, and when these conditions are encountered they must be followed. No road will be accepted which is narrower than 20 ft. wide on top of fills or narrower than 24 ft. between ditches in cuts, unless a lesser width is especially authorized.

In the case of roads to be surfaced, leave the rough grading perfectly flat in the center for a width equal to the width of the surfacing, plus 9 ft. On 9 ft. roads beyond the edge of this flat portion, the cross-section shall be rough graded exactly to the shape shown (on the standard cross-section sheet for earth roads) as proper for the soil and construction conditions encountered. This will make the subgrading work much easier as no dirt has then to be moved, except with the road machine, to make the trench for stone. It is best to do the grading and culvert work well in advance of the surfacing. That work is most successful on which the grading gang is out of the way of the gang laying stone or gravel.

For roads which are simply to be graded use the Cross Section for Earth Roads (shown in Fig. 1) which fits the conditions you are encountering. The change in cross-section for different soil conditions is very important, and the finished cross-section for 9 ft. macadam and gravel roads should be the same as the earth road sections for similar conditions. It is well to remember on all work that on fills about 2 ft. in width will usually be lost before the road finally settles, and to make fills wider than the final width called for.

Slopes in cuts must be hand trimmed to the slope required for the soil encountered. Easing up on work by leaving steep, untrimmed slopes is the poorest economy, and will not be allowed. Keep your ditch lines and edges of fills straight and true. Nothing will have more effect upon the looks of a job.

In the case of contracts for grading, the cross-section and slopes required must be specified in the body of the contract and if different cross-sections are to be used at different sections, so state, and give the cross-section numbers.

Guard Rail.—Guard rail should be placed wherever the drop from the fill to a landing place is more than 4 ft., unless slopes of 1 to 3 or easier are used.

Ditches.—In the final shaping up of the road with the road machine, be extremely careful not to dig in too deep in the ditches at any point so as to leave sumps to hold water, and if such places are made be sure to fix them so that they will drain out, either by further trimming with the road machine or by hand dug ditches. Nothing gives a worse impression than standing water in the ditches along a newly built road, and it is usually possible to avoid it both when newly built and thereafter if care is used in the final trimming up. No road except on marshes is completed until the ditches are so cut that they run clear a short time after rains.

Where roads across marshes are built, cut the ditches as far away from the center line as is reasonably possible (never less than 16½ ft.), and clear and grub the entire distance between the ditches. In some cases water will have to stand in such ditches, but bottom of ditch levels will be furnished by the division engineer wherever the fall is small but drainage is possible. The thing which should differentiate state roads from other roads is the securing of the best possible drainage in every case.

Tile Underdrain.—Those places which should be tile underdrained are usually designated on the plans, but some of these spots may be missed by the surveying engineer. Tile should be used in all cases where the road is naturally spongy or springy, or where running water is encountered in cuts. This holds true for roads to be only graded and drained as well as for surfaced roads.

Culverts and Bridges.—If any concrete or wood floored steel bridges lie on the road, surfacing material can be placed up to the ends, but in the case of wooden bridges where the new span will be over 6 ft. or where a steel bridge is to be replaced with a new one in a year or two, it is best to stop permanent construction 25 ft. from each end, and put in a temporary surfacing.

In staking out a culvert, establish the center line of the road from the stakes at least 200 ft. on each side of it, lay out one-half the total length of the culvert on each side of the center stakes nearest the culvert, and stretch strings between these last stakes. These strings will give you the inside edge of your outside end wall forms. Be super-careful to see that the end walls are parallel to the center line of the road. After lining up and setting up the end wall forms, sight along these in both directions and see that they look right and are right before starting to place the concrete.

Maintenance for Earth Roads.—The Com

mission has directed that in 1914 there be withheld from the amounts available for the construction of earth roads a sum sufficient to (1) keep the road dragged the balance of the season; (2) run over the road with the road machine the following spring; (3) remedy any erosions in fills or ditches which may have shown up during the first winter, and (4) provide for the dragging of the road the whole of the second season. The amount to be withheld will usually be about $100.00 per mile, though this will, of course, vary with the kind of soil and the length of the cuts and fills. In general, the heavier the original work the larger the maintenance fund necessary. It is absolutely necessary that the above means be taken to keep in good condition newly graded earth roads, and the cost of this work is a proper charge against the construction fund, inasmuch as no freshly graded road can be made a good road in one season. The towns will not properly attend to them, and it is inevitable that if they are not properly cared for in the months immediately following construction they will become (unless soil conditions are excellent) almost as bad as they originally were.

The best arrangement possible to make for dragging is to purchase a drag and to place it in the hands of some interested farmer living along or near the road to be maintained, and to pay him a fixed sum per mile for dragging the road. The common price is now $0.75-$1.00 per mile of road dragged for two round trips over the road. The man who does the dragging should also be given authority to correct any small defects as they show up and to keep the culverts open. A little care in arranging for the proper maintenance of dirt roads will furnish an object lesson to the community which may be of more value to it than the construction of the road itself.

The grading of sand roads without surfacing should be discouraged, unless it is intended to surface them at a very early date. It is, of course, proper to cut down hills and fill across marshes on sandy roads, and even where the surface of the new road is poor, hauling is usually better than on the old road, but the regrading of sandy roads where the grades at present are light is not beneficial, and it should be discouraged. Where sandy spots are encountered in grading any earth road, arrangements should be made to cover these sandy spots with clay and make a sand-clay road, or to cover them with gravel or some other surfacing material. It produces a very bad effect to grade a road, possibly spoiling the old clay surface in the process, and leaving the newly graded road almost impassable due to sand. Judgment in matters of this kind will be very much to your credit.

PREPARING FOR SURFACING.

(All instructions under this head apply equally well to stone, gravel, shale, or disintegrated granite surfacing.)

Width of Surface. Practically all surfaced roads will be built with a stone or gravel surface, 9 ft. wide, which experience in Wisconsin has proven to be ample for 90 per cent of even the main traveled roads. This width usually arouses some antagonism in a community until they have traveled the road for a few months, but if properly built and not humped up in a "hog back," the prejudice soon wears away and the people think the road "wide enough but not long enough." The 9 ft. road has won its way until it is the standard width in practically every county in Wisconsin which has been building stone or gravel road.

Roads near cities or villages with very heavy traffic will be built 15, 16, or even 18 ft. wide. If these are staked out for a 9 ft. road and the county highway commissioner or the town board thinks it must be wider to properly accommodate the travel, take the matter up with the commission. But remember always that even the dirt shoulders of the new road will probably be better at all times than the old road compared to it at the same season of the [illegible]

Point of Beginning.—Where a road is to be surfaced with crushed stone or crushed and screened gravel, it is best to commence work at the end of the road farthest away from the supply of material. The roller can then be run continuously without interfering with the teams delivering material.

On sandy soils it is best to modify this rule and build in sections of about 1,000 ft. at a time, starting 1,000 ft. from the end of the road nearest the source of material and working back toward the source. When this is completed, take another 1,000 ft. section, etc., driving over the completed sections and around the section under construction. By this method the road may be used as fast as completed; shoulders and subgrade will be kept in better shape; many cubic yards of surfacing material will be saved, and a far better road will be gotten than if the material being placed is hauled over continuously.

In those few cases where pit run gravel is used, and especially when used on a sandy subgrade, it is best to commence work at the end nearest the gravel pit, hauling over the first course for 1,000 ft. or more before laying the second course. Fill all ruts as they appear in each course immediately, either by the use of a small grader, a road drag, or, preferably, by raking in by hand.

Laying Out Subgrade or Trench.—Starting at the desired point, set two stakes opposite the reference stake, the distance between them being the width of the new road. To do this, refer to the grade sheet, which gives the distance from the side stake (placed when the survey was made) to the center of the new road. Subtract from this distance one-half the desired width of road and put in a stake with inside edge at this distance from the reference stake. Opposite this stake place another with its inside edge distant the width of the road from the inside edge of the first one. All stakes for subgrade should be made of ½-in. round iron about 24 ins. long, and about 25 should be kept on each surfacing job.

Stake out 700 or 800 ft. at a time. Be sure that the stakes are in line, except at bends or on curves. Usually curves will have to be staked out by eye to get good results.

Plow Work.—With a road plow cut as close to the inside edge of stakes as possible without disturbing them, turning the furrow toward the center of subgrade. Plow about 5 ins. deep. One furrow on each side is generally sufficient. Plow should be equipped with shoe or wheel and coulter. If a rooter is used, three furrows on each side will usually be necessary. Make first cut about 5 ins. deep as close to stakes as possible, the next 6 ins. nearer the center of the road. Drop the shoe down so rooter will run about 3 ins. deep for the third cut, working 6 ins. nearer center of subgrade than previous furrow.

Grader Work.—A light grader that can be handled with two horses is best. Use with the blade so set as to move the plowed ground from the center of trench or subgrade on to the bank outside of stake line. This work cannot be accomplished neatly with the grader alone, as some of the earth will roll back into the trench under the best of conditions. Make the trench deep enough. The depth at the sides should be at least the total loose depth of the two courses of material and more than this on sandy soils. It is much easier to throw out excess material with the road grader after the surface is laid than it is to bring extra material up from the ditches or to haul it in by wagons during the finishing when the trench has not been made deep enough to hold the stone. Nothing is more essential than a good solid shoulder, and the time to get it is before material is placed in the trench. In clay soils, after making the trench, plow drains through the shoulders every 50 ft. on both sides and every 25 ft. at low points between hills and immediately clean them out so they will drain the subgrade in case of rain.

The following procedure is not advised, as it is usually the most expensive way of getting shoulders. If the road has once been covered with crushed stone or gravel, and it is not desired to tear up the old surface, shoulders can be brought up to the stakes by bringing in dirt with the road machine from the side banks or from the ditches (if the latter material is fit to use), or can be hauled in with wagons. If the old road has a crown of 1 in. to the foot, it will take approximately 1,100 cu. yds. of compacted material per mile to build up 6-in. shoulders and retain the minimum width of 20 ft. on top. The cost of hauling and placing this material is usually very much greater than the value of the stone or gravel saved. As a matter of fact, no material is wasted if the subgrade or trench is cut in the old surface, the stone or gravel thrown out making an excellent shoulder. Failure is inevitable if an attempt is made to build a gravel or stone road with a heavy roller without first getting proper shoulders to support the material while it is being rolled.

Hand Work.—Straighten up stakes and drive them firmly. Tie a chalk or binder twine line to stakes on each side so line will draw on inside faces of stakes, drawing it tight. It is usually best to put in additional stakes at 50 ft. points so this line will not sag. These lines are to guide the laborer in trimming the shoulders so the edges will be straight and the grade uniform. On a 9 ft. road, if these lines are set 7 in. above center of subgrade, they will be 1 ft. higher than the bottom of trench at the shoulder. On a 15 ft. road set lines 6 ins. above center of subgrade. They will then be 1 ft. higher than the bottom of trench at the shoulder. The blade of an ordinary square point dirt shovel is 1 ft. long and can be used by the laborer to tell when trench is deep enough at sides by setting blade of shovel up to line. Clean out drainage trenches through shoulders, so that they really drain out from the trench. This will keep the trench from filling up in case it rains. It is well to widen the trench and road on the inside of curves, and to elevate the outer edge of curves.

Rolling Subgrade.—After the subgrade has been properly shaped to the same crown (or, better, a slightly greater crown) than the finished road is to have, it should be rolled until hard, especially if recently filled. Any hollows that develop during the rolling should be filled. Roll enough, but stop before the top layer of earth starts to slip. Wet spots in the subgrade should be shoveled out, filled with good earth or cinders and rolled. Don't leave sink holes with the expectation of filling them with crushed stone. They must be dug out and refilled with good material if a firm surface is to ever be gotten at that point.

CRUSHED STONE SURFACING.

Weights of Surfacing Material.—When buying by weight, to find the amount required, estimate that limestone weighs 2,500 lbs. per cubic yard of 27 cu. ft.; granite, disintegrated granite, and quartzite 2,800 lbs.; unscreened gravel or sand 3,000 lbs., and crushed gravel 2,650 lbs. Finer crushed sizes will weigh more than coarse sizes. The above averages the three sizes commonly used if they are dry. Screenings, sand, and gravel are often shipped wet and will then weigh far in excess of the above.

TABLE I—NUMBER OF CUBIC YARDS OF MATERIAL PER MILE TO MAKE A GIVEN LOOSE DEPTH FOR VARIOUS WIDTHS OF ROAD.

	Width of surfacing.				
	9 ft.	14 ft.	15 ft.	16 ft.	18 ft.
Depth of loose material in inches.	Cu. yds.	Cu. yds.	Cu. yds.	Cu. yds.	Cu. yds.
1½ in. (screenings)	180	280	300	325	367
3 in.	440	684	733	782	880
4 in.	587	913	979	1,043	1,174
5 in.	734	1,141	1,222	1,304	1,468
6 in.	880	1,369	1,466	1,565	1,760
Square yards of surface per mile	5,280	8,213	8,800	9,387	10,560

Spreading.—The simplest and most satisfactory way to spread stone accurately to a definite depth is to know how many cubic yards are in a load, and figure from Table II how many feet of road it should cover to the depth required, and to spread it over that many feet and no more or no less. It is much the most convenient to have all loads hauled the same size, in which case the spreader can measure accurately just how to dump each load. To find the capacity in cubic yards of a wagon, multiply the inside length by the width and then by the height (all in feet) and divide by 27.

It takes a good man to make a good spreader. Much of the looks of the road depends upon his work and much of the money available can be wasted by a careless workman who puts on too much material. Check the loose depth of the stone often and see that he is spreading each load to the proper distance so that stone is neither too thick nor too thin.

Daily Output.—For macadam roads, much of the stone will be locally crushed, usually with portable jaw crushers. The screen should be in two sections 4 ft. long, 30 ins. in diameter, the first section with perforations 1½-in. in diameter, and the second with perforations 2 ins. in diameter. The bin should have three sections with a total capacity of not over 25 tons, and should be portable.

Arrangements for stone supply to the crusher should be such as to permit crushing not less than 75 cu. yds., and preferably 100 cu. yds. per working day. A daily output of less than 75 cu. yds. means an expensive job.

On roads of crushed stone or on any road where a roller is at work all the time, the crusher and roller must work to capacity if the road is to be built cheaply. On these roads at least 75 cu. yds. of surfacing material should be placed each day, and if possible even more should be placed, up to 100 cu. yds.

When a one-course gravel road is built and the roller finishes it later, speed is not so essential, but on every road there is a certain fixed job cost and daily cost which is the same whether much or little material is placed daily. The most economical road is always the one built in the least working days. Work gangs of such size that they can produce a good output each day.

TABLE II—NUMBER OF LINEAR FEET OF 9-FT. ROAD A LOAD OF A GIVEN SIZE SHOULD COVER FOR VARIOUS LOOSE DEPTHS.

Weight of load.			Length spread for loose depth in inches.			
Granite.	Limestone.	Size of load, cu. yds.				
Lbs.	Lbs.		3 ins.	4 ins.	5 ins.	6 ins.
2,800	2,500	1	12 ft.	9 ft.	7.2 ft.	6 ft.
3,500	3,125	1¼	15 ft.	11.25 ft.	9 ft.	7.5 ft.
4,200	3,750	1½	18 ft.	13.5 ft.	10.8 ft.	9 ft.
1,900	4,375	1¾	21 ft.	15.75 ft.	12.6 ft.	10.5 ft.
5,600	5,000	2	24 ft.	18 ft.	14.1 ft.	12 ft.
6,300	5,625	2¼	27 ft.	20.25 ft.	16.2 ft.	13.5 ft.
7,000	6,250	2½	30 ft.	22.5 ft.	18 ft.	15 ft.
7,700	6,875	2¾	33 ft.	24.75 ft.	19.8 ft.	16.5 ft.
8,400	7,500	3	36 ft.	27 ft.	21.6 ft.	18 ft.

Foremen must compel spreaders to use this table

First Course.—(No. 1 stone, usually 2 ins. to 3½ ins. in size, usually spread to a loose depth of 5 ins., never more than 6 ins. and seldom less than 4 ins.)

The first course of rock or gravel can now be dumped on the subgrade and spread to the required thickness with rakes and shovels, after which it is to be rolled until it is packed firm and hard and there is no movement under foot as you walk on it. On very sandy soils, to keep the sand from working up through the stone, place a fair covering of clay, marsh hay, hay, straw, or weeds (good in the order named) before placing any stone. Shoulder trenches should be filled with the No. 1 stone so as to make blind drains. Cover the stone in the trench with sods, grass side down, or with hay or straw to keep dirt from clogging the water course. Care should be used in rolling the first course to keep the roller in the trench and not to overlap the stone and crush the shoulders. Roll from outer edge toward center. Clay or screenings are seldom necessary on the first course. Place about 200 ft. of first course stone before starting to lay No. 2. No. 1 bin does not usually hold this much, but No. 2 stone can be put in the bottom course until the first course is laid 200 ft. ahead provided the two sizes are kept in separate strips, laying say the first 125 ft. of No. 1 and then starting with No. 2. Some prefer to pile the No. 2 until the No. 1 is far enough ahead and use it when the end of the work is reached.

Harrowing.—Although not much used in Wisconsin, many highway engineers claim that a good stout spike-toothed harrow is a necessity in properly compacting stone and gravel, and that a large part of the rolling of the two first courses can be saved by thorough harrowing after spreading.

Second Course.—(No. 2 stone, usually ½-in. to 2 ins. in size; usually spread to a loose depth of 4 ins., never more than 5 ins., seldom less than 3.)

The second course of stone is now hauled on top of the first and spread evenly to the depth called for. This course is rolled commencing on each outer edge with the rear wheel half on the stone and half on the shoulders and rolling toward the center. The roller should be run over this course a number of times until the stone or gravel is brought to shape and fairly well compacted before screenings or dust is applied. All low places that have shown up should be filled with No. 2 stone and high spots raked down so that the surface presents a smooth, even appearance.

Third Course of Screenings.—(No. 3 stone, usually from dust to ½-in. in size. Not necessary in pit run gravel or shale roads.)

Screenings may be hauled at any time after subgrade has been finished. Dump in piles with inner edge about 2 ft. from edge of trench or subgrade. Never dump screenings on the second course. If a cubic yards of 27 cu. ft. is dumped in one place, the centers of piles should be about 30 ft. apart for a 9 ft. road, or about 18 ft. apart for a 15 ft. road. A cubic yard of screening under ordinary conditions will cover about 275 sq. ft. of surface. A cubic yard of screenings bought by weight is not a cubic yard by measure, especially if wet.

Use a square point dirt shovel to apply screenings. Put them on very thinly with a quick sweeping motion, working from one end of the road toward the other . Keep roller running constantly while putting on the screenings. Repeat this process until all voids between stones are filled. Roll until stone stops moving and the surface is hard, and all second course stone slightly covered with screenings. It is necessary to bind quartzite or granite with some good cementing material, such as clayey pea gravel or disintegrated granite. If neither of these are available, it is best to use a limestone top course or bind the granite with a bitumen. Clays and limestone screenings have not proven to be satisfactory binding materials for this class of stone. If clay is used, apply it dry a little at a time, and after the voids are well filled and the road is rolled, cover the surface with screenings, preferably with the dust screened out. Don't use too many screenings—just enough to nicely cover the road at all stages is just right. Too many screenings simply allow the road to rut more easily.

Flushing.—With the stone covered with screenings the road is ready for water. On clay soils trim down shoulders with road machine before applying water to prevent too much mud at sides. Don't try to finish over 400 ft. of road at one time. It will dry out before you get it soaked. It is customary to finish up each night or every other night the road ready for finishing. If screenings stick to tires, scrape them off or keep tires well wet or both. Sprinkle road until screenings are thoroughly soaked and tires run clean. When road is wet enough, follow sprinkler with roller. If screenings pick up on roller wheels, stop rolling and apply more water. Keep roller as close to sprinkler as possible. If bare spots or holes appear, put on enough dust to cover them. Sprinkle and roll until water is continually carried along in front of the roller wheels at every point of the road being flushed. Road is then finished. Pit run gravel roads are flushed in about the same way, but must be very wet when rolled or quite dry. A rainy day is the best time to finish a road.

If the road fails to compact and is spongy under the roller, the subgrade is too wet. Take the roller off and wait until the road dries out, or if conditions are very bad, dig out the stone and sand underneath, refill with good dry material and surface again. Use tile drain where necessary. Never have more than 600 ft. of unfinished road on your hands. Keep the work compact and finish as you go.

Trimming Up.—After the road has dried sufficiently, use a road machine to smooth off the shoulders to proper slopes. Trim shoulders so water can get from center of road to ditches at every foot of the road's length. Remove all surplus material. Roll shoulders as far out as is safe to take roller and trim off the excess dirt remaining. Road is then finished and can be thrown open to travel. Whenever the ground permits, it is a good plan to smooth off and widen one or both of the ditches so as to permit teams to use them as a dirt road in dry weather and as snow roads.

Surface Treatments.—It is very necessary to place on macadam and gravel roads a protective coating of some bitumen to prevent excessive wear and the unraveling incident to heavy automobile traffic. It is best to apply these bituminous coatings some months after the waterbound macadam or gravel road is built. The ordinary treatment consists of sweeping the roads entirely free of dust, horse droppings, and other dirt so that the stones are exposed as much as possible without loosening them. It is then wise to thoroughly wet the road, thus further cleaning it, and after it has dried out, to apply the bitumen. The bitumen should consist preferably of a refined tar, or an asphaltic oil containing at least 80 per cent of asphalt. Paraffin base oils should not be used, and the application of oils containing less than 80 per cent of asphalt (usually known as light oils) is not advisable.

The bitumen should be applied at the rate of about one-half gallon per square yard of surface. Some types of bitumen are simply applied, left 24 hours or more to thoroughly soak in, and the road then thrown open to travel. Other types of bitumen are applied as above and immediately, or a few hours after application, there is applied a coating of sharp, clean sand or screenings (preferably with the dust eliminated) to absorb the surplus bitumen. The road is then either rolled and thrown open to traffic or thrown open to traffic without rolling. The cost of applying surface treatments of bitumen will vary from 5 to 10 cts. per square yard of surface treated, the variation being the difference in the price of the bitumens and the difference in other conditions. An advisable procedure for roads built in 1914 is to throw them open to traffic until the early part of next summer. Surface defects should then be corrected and the bitumen applied as above.

OTHER SURFACES.

Gravel.—Gravel roads built of gravel just as it occurs in the pit can be built in one course and fair results secured if no roller is available to roll it until after all material is placed, but it makes a much better road if the gravel is laid in two courses of 5 or 6 ins. each loose, and each course thoroughly rolled. If no roller or traction engine is available for the first course, make the trench and commence to lay and spread gravel commencing at the end from which the gravel is hauled, hauling all gravel for the road over the first course. If this makes hard hauling, apply clay until the gravel packs, and keep the ruts raked full. After hauling a good stretch of the first course, laying second course can commence and the top course rolled when a roller is available. No stone larger than 4 ins. should be allowed in the bottom course, and none larger than 2 ins. in the top course. Always rake the larger stone to the bottom of the course being placed. Rake thoroughly.

The foregoing is not the advised procedure, and roads will not be so constructed except when permission is given in writing by the division engineer.

The Commission prefers to have pit run gravel roads built just as stone roads are built except that no screenings or third course is required. Crushed gravel should be laid in three courses in exactly the same manner as crushed stone. When at all doubtful as to the advisability of using a gravel as it comes from the pit, notify the State Commission and an inspector will be sent to look it over with you and to determine how best to handle it. Crushing and screening, screening out sand or excess clay, or adding clay and harrowing it in will often work wonders with a gravel that does not pack well.

Shale.—In a few counties shale roads are being built. Shale must be laid in a trench and in two thoroughly rolled courses to get the best results. A slight covering of clayey gravel will produce a much better wearing surface than the shale alone.

Sand-Clay.—Roads built by placing clay on sand and thoroughly mixing them, or vice versa will give good results when properly built.

Bituminous Macadam.—The construction of bituminous macadam roads has not been encouraged by the Commission. In especial cases macadam roads should be built with the top course of stone mixed with the bitumen either by the penetration method or the mixing method. This will be especially true where granites or quartzites are used as second course material and good binding material is not available at a reasonable cost. Experience has demonstrated that the harder rocks bound with clay have not proven to be a satisfactory surfacing under heavy automobile traffic, and the tendency will undoubtedly be to build such roads only when good binding gravel or a bitumen is used as a binder for the second course. Where county highway commissioners have the problem of constructing macadam roads with stone other than limestone, it would be advisable to consider thoroughly this matter of bituminous surfacings before binding with clay.

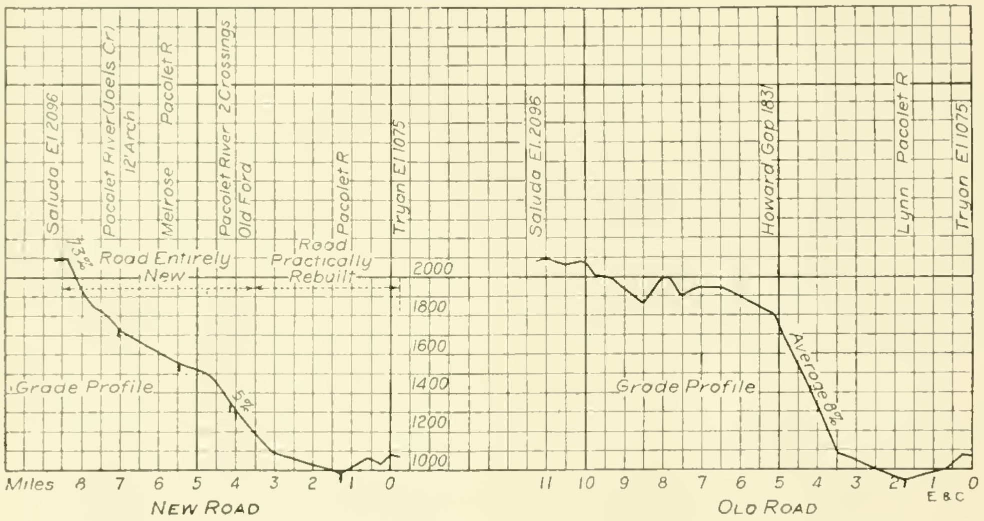


Fig. 1. Comparison of Profiles of Routes Between Tryon, N. C., and Saluda, N. C.

Concrete.—Concrete is a coming type of construction for heavily traveled roads and for roads subject to overflow, especially when the overflow is rapid. Don't build any stretch of concrete road without instructions from the state commission as to how to build that especial piece and what materials to use. To be successful they must be properly built of good, clean, screened material, and of a mixture not leaner than 1 part cement, 2 parts sand and $3\frac{1}{2}$ parts clean gravel or crushed stone not larger than 2 ins. in size. Concrete roads will not be built of pit run gravel except upon permission from the state highway engineer.

The ordinary cost will be about $1.20 a square yard, although it may vary from 90 cts. to as high as $1.75.

GENERAL NOTES.

Care of Machinery.—Road machinery is an expensive investment, but doubly so if care is not exercised in its maintenance. If a few simple and inexpensive precautions are observed, it will result in an annual saving of thousands of dollars to the counties.

In the case of rollers, rock crushers and crusher power, which during operation are constantly exposed to dust, a liberal supply of oil or grease should be used to flush all bearings and thus prevent the rapid wear resulting from the presence of grit and foreign matter in dry bearings. Good lubrication of all machinery is the first step toward its economical maintenance.

Keep a good supply of extra parts on hand in case of breakage, especially for rock crushers. An extra link for the elevator chain may save many valuable hours. Extra crusher dies should always be on hand.

A tool box equipped with waterproof cover and fastened with a padlock is a necessity. This box should contain, among other things, a good supply of bolts of assorted sizes and lengths, with washers to match; also rivets, screws, nails and spikes. In brief, keep constantly on hand a generous supply of everything likely to be needed in case of a breakdown.

Insist that the roller operator keep his machine clean and in good repair. All grease and dust should be removed from the machine sometime before the close of each day's work. A roller that has the appearance of a piece of junk after a single season's work is evidence of inexcusable neglect on the part of the county highway commissioner, foreman and roller man.

On the larger jobs, a small portable blacksmith's outfit is very useful. With this equipment, rooter and plow points and small tools can be sharpened as often as may be necessary without the delay incident to sending them to a blacksmith. Dull tools make expensive work.

When the season's work is completed, all machinery should be cleaned thoroughly, and a coating of heavy oil applied to all polished surfaces, boiler jackets, shafts, etc., thus preventing rust. Also paint smoke boxes and stacks. Machinery should be stored under cover. It is not necessary that storage sheds have tight sides, but a good roof is essential. It is not the usual practice to keep rock crushers under cover, but if this is done the machine will be in better running order when work is again begun. Rollers, road machines, power engines, wheelers, slips, plows and sprinkling wagons should always be protected during the winter season. Drain all water from all machinery before leaving it.

Reminders.—Have all loads hauled of the same size. It helps to make spreading easy.

Keep traffic off the road surface while it is being built. A road can not be cheaply or satisfactorily finished with teams constantly passing over the stone.

Don't lay stone in the mud; wait until the road dries out. If side drains are properly made and are kept open, the subgrade will clear of water immediately after rains, leaving it dry and hard.

No good stone or gravel road was ever built without a good solid subgrade to start from.

Don't roll first course of stone too much. If dirt comes up so it shows, stop rolling.

Don't be afraid to roll the second course after you commence to apply screenings. The more it is rolled the better the road will be.

If the stone crawls ahead of roller, roll very slowly. In general, it is best not to run roller too fast.

Put screenings on evenly and a thin layer at a time—the roller will work them into the spaces between the stones.

Don't forget to grade whenever possible a dirt road in the ditches of surfaced roads. It will be much used in dry weather.

The foreman who makes a reputation as a road builder is the one who has the capacity of taking care of men and tools, who knows what good work looks like and does it, and who is content with no work that he does unless he has done his best.

Attention to the little details that seem hardly worth while will make the difference between a fine road and just a road.

Location and Construction of a Section of Mountain Road Between Asheville, N. C., and Spartanburg, S. C.

Contributed by Albert W. Merrick, M. Wn. Soc. C. E., Boone, Ia.

Mountain road construction is interesting because of the unusual features of location generally presented and the heavy excavation usually necessary. The great expense involved in constructing mountain roads makes any data on roads already built, or in process of construction, of great value to the engineer or contractor engaged in this class of work.

The road between Asheville, N. C., and Spartanburg, S. C., a distance of about 60 miles, was completed about a year ago, with the exception of a section $8\frac{1}{2}$ miles long between Tryon and Saluda in Polk Co., N. C. The location and construction features incident to the finishing up of this section are described in some detail.

Location.—Some idea of the difficulty in locating either highways or railways in this section may be had when it is known that the maximum grade of the Southern Railway in this locality is 4.8 per cent. A natural outlet for a highway is provided by the Pacolet River which rises near Saluda, flows eastwardly and passes just north of Tryon. This outlet had not previously been taken advantage of on account of the rocky and precipitous walls of the valley which in many places approach close to the stream.

In determining the location two routes were considered: (1) by way of Howard Gap, where a road already existed, and (2) a valley location by way of the Pacolet River. About three miles of the lower end of the Howard Gap route could have been entirely relocated so as to secure a maximum grade of about six per cent, but this would have added one mile to the distance between Tryon and Saluda. The valley or Pacolet River lo-

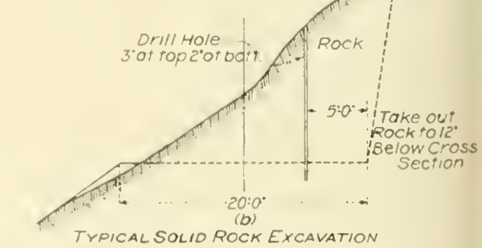

1 to 1
1½ to 1
Sub Grade
20'0"
(a)
CROSS SECTION IN EARTH
Drill Hole 3" at top 2" at bott.
Rock
5'0"
Take out Rock to 12" Below Cross Section
20'0"
(b)
TYPICAL SOLID ROCK EXCAVATION

Fig. 2. Details of Construction Showing: (a) Cross-section of Road, (b) Placing of Holes for Blasting.

cation involved the construction of five miles of new road with heavy rock cuts and four bridges across the Pacolet River. A fairly good road already existed for a distance of 3½ miles from Tryon to an old river ford. From this point relocation and heavy rock excavation was necessary. Grade profiles of the two routes are shown in Fig. 1. The Pacolet River route was adopted.

In locating the adopted route improvements in alignment and grade were introduced in the portion along the river. The relocation from the old ford to Saluda is along a rock cliff, the road being a shelf cut in the rock. The cross-section adopted is shown in Fig. 2(a).

Construction.—In making the rock excavation, drill holes are located about 5 ft. from the inside line of the finished roadway and about 8 ft. apart longitudinally, as illustrated in Fig. 2(b). The holes are 3 ins. in diameter at the top and 2 ins. at the bottom. In blasting, the hole is sprung with 1½-2 sticks of 40 per cent dynamite and loaded with about four keys (100 pounds) of black powder. Any irregular points of rock which may be left are broken off with small charges of dynamite. Most of the rock excavation is in gneiss. After blasting (except in ledge rock) the rock usually stands at a slope of ¼:1. The character of the rock excavation is shown in Fig. 3. A Sullivan steam drill is used, supplied by a portable horizontal boiler. Negro labor is used for breaking up small stone, wheeling dirt and refuse and cleaning up after blasting. For some of the heavier cuts in earth a revolving steam shovel is used.

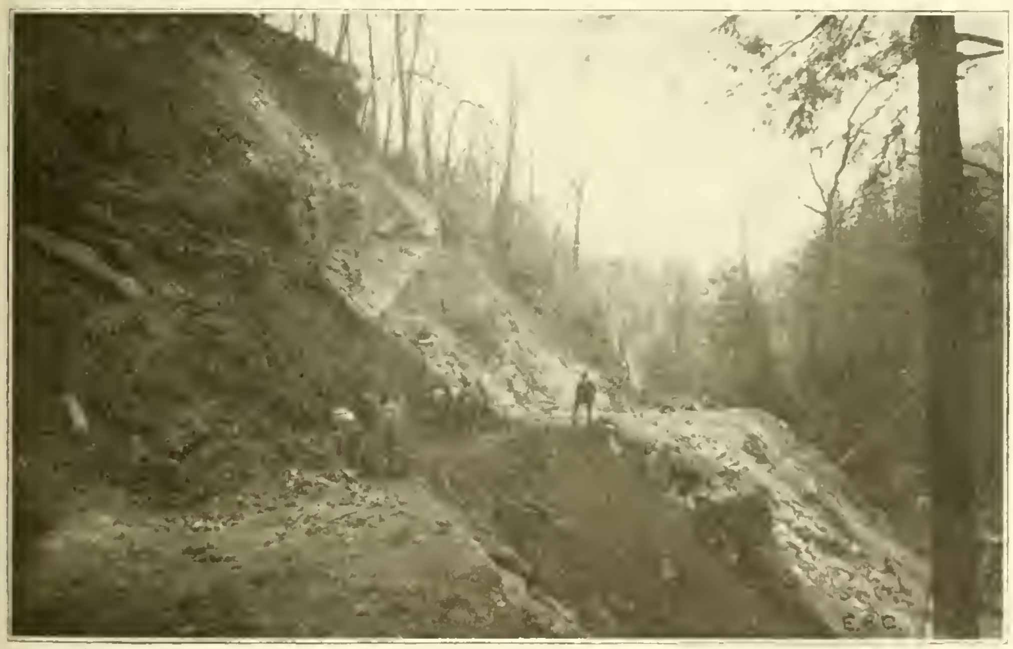


Fig. 3. View Showing Type of Rock Excavation.

Natural rock quarried and cut roughly to dimension on the site is used for all box and arch culverts and abutments for steel bridges. At one bridge site the rock ledges forming the banks of the river were blasted smooth and used for abutments. The four 50 ft. bridges for river crossings have riveted, pony trusses and a clear roadway of 16 ft.

The road will be surfaced with sand-clay. On sections of the road having a clay base, 6 ins. of sand will be applied and thoroughly incorporated with the underlaying clay. The surface will be rolled. On the rock sections both sand and clay will be hauled. This will be mixed and rolled to form the surface. A sum of $10,000 is available for the work of surfacing.

The location and construction of the road is in charge of Mr. S. C. Cromwell. Mr. W. L. Webster, resident engineer, is located at Tryon, N. C. The contract for grading and bridge work (except steel bridges) was let to Mr. C. D. Greer.

Notes on Street Lighting for Small Towns.

In the construction of pavements in small towns the road engineer is sometimes called upon to give an opinion upon the method of lighting to be used and the placing of the lamp posts. Mr. A. P. Little, assistant professor of electrical engineering in the Oklahoma Agricultural and Mechanical College, Stillwater, Okla., in his paper, "Street Lighting for Small Towns," has discussed these questions in a clear and elementary manner, and a portion of his paper is given here.

A small town should not attempt to rival a large city in its street lighting. In towns and villages the density of population is less and the value of real estate lower than in cities. Hence, if the same degree of illumination is attempted, the cost per capita and the ratio of the value of lighting equipment to property value may become excessively large.

Choice of Illuminants.—The choice of illuminants lies between tungsten incandescent lamps and arc lamps. The carbon filament lamp is obsolete for this purpose because of its low efficiency, yellow light, rapid decrease in candlepower with age, and its short life. The average life of the series tungsten is about 1,000 hours, while that of the carbon filament is 600; the efficiency of the tungsten is about .85 candlepower per watt; that of the carbon .3 candlepower per watt. Of the arc lamps the open D. C. arc is now seldom used, although if kept in proper trim it gives better results than the A. C. enclosed arcs which have largely superceded it. The new types of arc lamps—magnetite flame and carbon flame—are being rapidly introduced and it is not unreasonable to expect that these will to a large extent replace the older types of carbon arcs. The nitrogen filled tungsten lamps recently brought out, which give two candles per watt (2½ times as efficient as the ordinary tungsten), will no doubt come into extensive use for street lighting.

Placing of Lights.—The method of lighting which is least expensive in first cost and maintenance is by means of tungsten lamps bracketed from poles at street intersections. Overhead suspension at street intersections gives better distribution of light but costs slightly more. On business streets, where there is considerable traffic, a 100 watt tungsten is about the smallest that can be used. Two hundred and fifty watt lamps give very satisfactory service unless they are spaced too far apart. On residence streets, lamps as small as 25 watts supported on brackets from poles at each street intersection have been used, although lamps of less than 75 watts do not give good results. If the town is ambitious to have more artistic lighting on its main thoroughfares, closer spacing of the lights is necessary. These lights may be mounted on brackets, as in the plan outlined above, or better still, on ornamental posts. Overhead wires are hardly permissible in ornamental lighting, therefore the wires should be run underground in conduit.

From the standpoint of economy of power, small units and close spacing are desirable. For example: One hundred watt lamps spaced every half block will give the same illumination midway between lights as 400 watt lamps spaced a block apart, because the light intensity varies as the square of the distance and the area to be lighted is a long narrow strip. The point at which close spacing ceases to be economical is reached, however, when the cost of maintenance plus the increase in cost of interest, depreciation, etc., on investment exceeds the saving on the reduced power cost. No definite rule can be given to determine the economic spacing, because the cost of power, material and installation will vary in different localities. Each design must be worked out separately.

For very bright illumination of streets, arc lamps will be found very satisfactory, especially the high efficiency arcs or nitrogen filled tungstens. These should be placed high above the streets, not less than 20 ft. from the pavement, and for the most artistic effect should be mounted on ornamental posts. By short or long spacing of the arcs in the block, any degree of illumination may be obtained.

Elimination of Glare.—It is very important to eliminate glare as far as possible. Glare occurs when a bright source of light is directly in the line of vision. The eye naturally adjusts itself to the brightest light in the field of view, and consequently objects in the shadow are difficut to perceive. It does not seem to be possible to avoid it entirely, but it may be reduced in the following ways:

(1) By enclosing lamps in opalescent globes.

(2) By placing lamps high above the street.

(3) By placing the lamps in a line with the street curbing.

(4) By using shades or reflectors of proper design.

By combining these methods as far as practicable the best results will be obtained.

Systems of Distribution and Control.—The increase in illumination in a growing town or village is usually gradual—first a few lights in the business districts, then a few on the residence streets, gradually increasing the number and size of the lamps until quite a respectable street lighting system is built up; respectable, at least, in point of size. At first these lamps are supplied with current from house-lighting transformers. If the central station is not giving day service it will not be necessary to turn them off at any time. If the plant runs during the day, as well as at night, provision should be made to turn them off and on, although this is not always done. For instance, to burn an 80 watt lamp during the day from dusk to dusk will cost on the average of 4 cts. For the equivalent of say fifty 80 watt lamps, this would amount to $2.00 per day, which is much more than it would cost to turn them on and off each night, even with individual switches. By controlling several lamps from one switch, considerable labor will be saved.

For a small system of a few dozen lamps the parallel scheme of operation may be simplest and most practical. In comparing the series and parallel system it is well to note that the quantity of copper required varies inversely as the square of the voltage employed. For instance, to deliver a given amount of power at a certain per cent loss with 100 volts will require 100 times the weight of copper that is needed to accomplish the same result with 1,000 volts. Therefore, for any considerable number of lamps the series system is most economical, since the constant potential system is essentially low voltage, while the series system may employ voltage from 500 to 5,000 or more.

Posts.—There are a number of ornamental posts on the market, from the simple single tungsten unit to the post with several arc lamps. A three to five lamp post using 40 to 100 watt lamps each enclosed in a crystal opalescent globe presents a rather massive appearance and is especially suitable for wide streets. For "white way" effects there should not be less than four posts to a block. If single lamp posts are used, more will be required to the block for good illumination, or higher candlepower units may be used.

The tungsten posts are ornamental in daytime as well as at night, and their chief disadvantage is that the glare is somewhat disagreeable because the lights are usually placed low to avoid excessive cost of posts. Tungsten posts with several lamps have the advantage that they cast no shadow as do single arc lamps. On streets where arc lamps placed high would give excessively deep shadows because of the dense foliage of the trees, the tungsten posts may be used to advantage.

Main St., Stillwater, Okla., furnishes a good example of ornamental street lighting. This street is lighted with eights posts to the block, each post having three 100 candlepower tungstens, each enclosed in opalescent globes. This is the equivalent of six enclosed arc lamps to a block and gives excellent results.

Drainage and Irrigation

The Duty of Water.

Contributed by Louis Schmeer, Long Beach, Cal.

The term "Duty of Water," denotes:

(1) Either the quantity of water required per season to irrigate a unit area of land—one acre, one hectare; or

(2) The area irrigated per season by a continuous flow of a unit of quantity—second-foot, second-liter;

(3) The quantity continuously flowing which is required to irrigate a unit area of land per season—fraction of a second-foot per acre, second-liters per hectare.

In the first case, the units of quantity are the acre-foot and the hectare-centimeter, quantities sufficient to cover the area of one acre one foot, the area of one hectare one centimeter deep.

Since the area of one acre is equal to 43,560 sq. ft., one acre-foot contains 43,560 cu. ft. = 325,850 gals. = 1,360 tons of water. Likewise, since the area of one hectare is equal to 10,000 square meters, one hectare-centimeter contains 100 cubic meters or 100 metric tons.

In the second case, since there are 86,400 seconds in 24 hours, a flow of 1 sec.-ft. is equal to 1,983 acre-feet per day or 724 acre-feet per year. Likewise a flow of one second-liter is equal to 0.864 hectare-centimeters per day, 315.36 hectare-centimeters per year.

In the third case, the proper unit of quantity is 0.01 sec.-ft., the customary unit a miner's inch equal to 0.02 sec.-ft., the legal unit a miner's inch equal to 0.025 sec.-ft. in California, Montana, Arizona, etc.

Since 1 sec.-ft. per year = 724 acre-feet, 0.01 sec.-ft. per year = 7.24 acre-ft., a customary miner's inch per acre per year = 14.48 acre-feet, a legal miner's inch per acre per year = 18.1 acre-feet in California, etc.

THE NET DUTY.

The net duty represents the quantity of water actually used in the production of crops; the gross duty includes also the quantities lost through seepage and evaporation in canals and ditches.

TABLE I.

Description of plants.	Number of trials.	Pounds of water required to produce 1 lb. of dry matter.	Fraction of acre-foot of water required to produce 1 ton of dry matter.	Fraction of acre-foot of water required to produce 1 bushel of grain or potatoes.
Wheat	[illegible]	453	0.333	0.025
Barley	[illegible]	464.1	0.341	0.0223
Oats	[illegible]	503.9	0.369	0.0132
Corn with ears	52	270.9	0.199	0.0140
Peas	1	477.2	0.351	.....
Potatoes	14	385.1	0.283	0.0175
Clover hay	16	576.6	0.423	....
Decid. trees		510	0.375	.....

The water requirements of crops under various conditions of culture have been closely studied. Hellriegel in Germany and King in Wisconsin have done valuable work along this line of research; many important facts have also been brought to light by the U. S. Department of Agriculture. Hellriegel and King undertook both pot culture and field experiments and determined the loss of water with great precision.

In Table I, based mainly on King's experiments, Column 3 gives the quantities of water in pounds required to produce 1 lb. of dry matter; Column 4 the fraction of an acre-foot required to produce one ton of dry matter; Column 5 the fraction of an acre-foot required to produce one bushel of grain or potatoes. In the experiments on which this table is based no water was lost either by surface run-off or by percolation beyond the reach of the roots of the plant.

TABLE II.

Description of soil.	Pounds of water required to produce 1 lb. of dry matter.	Fraction of acre-foot of water required to produce 1 ton of alfalfa.
Clayey loam	559	0.41
Very shallow red clay soil	1,129	0.83
Clayey gravel	1,024	0.79
Clayey loam with spots of gravel	1,129	0.83
Gravel with a slight cover of silt	2,412	1.80
Gravely soil	3,405	2.50

Although the data given are based on actual field experiments, they represent cultural conditions more favorable than those usually met with in practical agriculture and the quantities of water indicated are to be regarded as the minimum amounts necessary to bring a crop to maturity.

The data in Column 4 indicate that slightly more than one-third of an acre-foot is required to produce one ton of the cereals, including straw and stubble, under the most favorable conditions. The lesser quantity required by corn is no doubt due to the thorough cultivation this plant receives.

TABLE III.

	Water applied in inches.	Initial moisture per cent.	Evaporation in inches.
Loss, in inches, first 5 days, uncultivated	11.9	Top 12-in. dry, 6 per cent in balance of soil	1.76
Loss, in inches, in next 6 days, uncultivated			1.39
Loss in corresponding 6-day period, cultivated			0.51
Loss, in inches, first 3 days, uncultivated	8.0	Top 4-in. dry, 6 per cent in balance of soil	0.84
Loss, in inches, in next 3 days, uncultivated			0.29
Loss in corresponding 3-day period, cultivated			0.10

The amount of water required to bring a crop to maturity depends upon many factors. Of paramount importance are the composition, texture and depth of soil and subsoil, the method of cultivation, the method of applying the water, the distribution of the rainfall and to some extent the peculiarities of the plants.

Table II based on measurements of water delivered to alfalfa fields in California, gives data indicating the great influence of the depth of the soil and its texture on the amount of water required.

On average soils in the state of Washington the amount of water required to produce one ton of alfalfa hay has been found to vary between 0.365 and 0.45 acre-foot, in New Mexico 0.428 acre-foot.

Perels found the relative water requirements of different soils to be as follows:

Very heavy soils	1.00	....	..
Heavy soils	1.15	1.00	....
Medium soils	1.68	1.46	1.00
Light soils	2.30	2.00	1.36

It is plain that the water requirements of a soil to produce a given quantity of matter will be the less the greater the fertility because the loss due to evaporation, percolation or surface run-off will necessarily be relatively the less the greater the quantity of matter produced.

Cultivation after the application of water just as soon as the topmost 3 to 6 ins. of the soil become dry enough to be converted into powder and frequent and thorough cultivation thereafter, reduce losses due to evaporation and conserve the water contents of the soil. The object of cultivation is the destruction of the capillary action by depriving the surface layer of the soil of its moisture: for capillary water can not ascend into zones entirely dry and consequently can not reach the surface and evaporate.

The effect of soil cultivation on the loss due to evaporation was investigated by Fortier and the results of his experiments are summed up in Table III.

In Table IV, also by Fortier, the figures in columns 2 to 6 illustrate the influence of a soil-cover on the loss due to evaporation. The depth of water applied was 3.2 ins. After this had sunk in, the soil was covered to the depths indicated in Column 1.

INFLUENCE OF METHODS OF IRRIGATION ON THE DUTY OF WATER.

There are many ways of applying water to the soil and the amount required to bring a crop to maturity depends to some extent on the method in use.

Sub-surface irrigation, apparently the ideal method of applying water, is in most situations impracticable. It is a success only in very porous soils in which the water table is at no great distance from the surface and the soil always contains a good percentage of capillary moisture. The method was formerly much in vogue in Europe but was found to be inefficient. In some localities it is still used on a small scale.

TABLE IV.

	Evaporation in inches				
Depth of cover.	First 3 days.	Next 4 days.	Next 4 days.	Next 3 days.	Total for 10 days.
No mulch	0.43	0.19	0.08	0.02	0.82
4-in. mulch	0.13	0.03	0.03	0.02	0.21
8-in. mulch	0.04	0.01	0.02	0.01	0.08
12-in. mulch	0.01	0.00	0.01	0.00	0.02

Furrow irrigation, principally used in orchards, vineyards and truck gardens, if properly applied, is one of the most economical and efficient methods in use. Fortier found the loss due to evaporation to depend in a great measure on the depth of the furrows through which the water flows and gives the following data. Table V based on experiments made in California.

Check irrigation, basin irrigation, border irrigation is the prevailing method of irrigating meadows, pastures, clover and alfalfa fields and, in some localities, orchards and vineyards. It is also applied in the culture of rice, sugar cane, etc. Where this method is in use the field is divided into plots denoted checks or basins. These are flat reservoirs bounded by temporary or permanent levees or dikes. In meadows, pastures, grain and alfalfa fields, the principal dams run along contour lines and the plots are 0.25 to 1 ft. apart vertically: subdivision dams run with the slope. Temporary rectangular basins of small area are used to irrigate orchards and vineyards.

TABLE V.

Method of application.	Evaporation in a period of 10 days.
Water applied at the surface, no furrow	Loss 25 per cent.
Water applied in a furrow, 3 ins. deep	Loss 21 per cent.
Water applied in a furrow, 6 ins. deep	Loss 17 per cent.
Water applied in a furrow, 9 ins. deep	Loss 16 per cent.
Water applied in a furrow, 12 ins. deep	Loss 12 per cent.

Sheet irrigation is a method frequently used to irrigate alfalfa fields. Where this method is used the land is divided into long rectangles by low dams running with the slope. The water is admitted into the beds at the upper end from ditches running along contour lines in sections and flows down the slope in thin sheets.

Contour channel irrigation is a favorite method in Europe, especially in the South, of irrigating meadows and clover and alfalfa fields. The method is similar to that described last: the supply channels run along contour lines in sections, the water passes over the edges and flows in thin sheets down the slope, which is usually 2 to 4 per cent. The distance between distributors varies between 25 and 75 ft. The plots are not divided by levees as in the sheet flooding system.

Bed flooding, another method much in use in Europe is a combination of the basin irrigation and sheet flooding methods. A stream of water flows constantly into a basin at one end, the surface of the bed is covered to a depth of 3 ins., the surplus passes out at the other end into a waste channel. It is claimed that by the application of this method the fertilizing contents of the water are precipitated to a greater extent than where contour flooding is used.

Wild flooding is a system of irrigation in use where the land has not been prepared for the distribution of the water. It is the most wasteful method. Water is admitted in streams at the highest points in the field and no attempt is made to effect an equal distribution.

Pipe flooding is a method in vogue in the alfalfa fields of Southern California. The water arrives at the head of the field in a cement pipe and flows through a hydrant into a 6 or 8-in. slip-joint light steel pipe lying loose in the field. From the end of this the water flows into the stubble. The distribution begins at the head of the field and sections of the pipe are slipped on as the distance from the hydrant increases. This method of distribution is efficient but costly, no expensive preparation of the land is, however, required.

The seasonal distribution of rainfall is another factor having some influence on the quantity of water required to mature a crop. It has been observed that in regions where summer rains prevail and are frequent the root system of the cultural plants has a tendency to develop near the surface. Since the surface layers are most exposed to the evaporating effects, they and the roots embedded will lose their moisture easily and the plant will suffer unless frequent showers occur. West of the Rocky Mountains where winter rains prevail and no rain falls during the last part of the growing season, the roots recede from the threatening drought and heat, growing downward toward zones where the moisture accumulated during the winter is in store. The plant is in consequence but little affected by conditions existing at the surface and will mature even if no rain falls during the last 60 days. To this is due the fact that a crop will mature west of the Rockies with a less abundant supply of moisture than on the great plains.

Table VI based on data published by the U. S. Department of Agriculture, gives the net duty of water for some of the common agricultural plants under conditions existing in the different states of the West.

In Southern California the usual allowance for citrus groves is one miner's inch (0.02 sec.-ft.) continuous flow for every five acres. This corresponds to a duty of 2.23 acre-feet

METHODS OF ESTIMATING THE NET DUTY OF WATER.

In estimating the net duty of water, the quantities required to produce one ton of hay, one cental or bushel of wheat, barley, oats, potatoes, etc., may serve as the basis for calculations.

The figures given in Table I above show that an average of one-third of an acre-foot is required to produce one ton of dry matter in the case of the cereals, provided there is no loss due to percolation and surface run-off. This holds also good for alfalfa. In practical agriculture these losses can rarely be entirely avoided. From experimental data given by the Department of Agriculture it appears that the average amount of these losses from compact, loamy or clayey soils is about 1/12 acre-foot per ton of dry matter produced. Consequently the actual amount required to produce one ton of dry matter in these soils is $\frac{1}{3} + 1/12 = 5/12 = 5$ acre-inches or 0.417 acre-foot.

Although the texture of the soil has no direct influence on the actual water requirements of the plants themselves, it affects the quantities which may be retained within the reach of the roots of the plants. From clayey and loamy soils there is little or no percolation but the evaporation losses are a maximum if cultivation is neglected. For light sandy and gravely soils the reverse holds good. The percolation losses far surpass the evaporation losses from compact soils and the combined losses will in consequence increase with the looseness of the soil.

From data at present available it appears that the relative water requirements of soils of different degrees of perviousness are about as follows:

1. Average clayey or loamy soils 1.00
2. The same but shallow 2.00
3. Medium soils, sandy or gravely soils with a good percentage of clay or loam 1.50
4. Light soils, sandy or gravely soils with a small percentage of clay or loam .. 2.00
5. Loose soils, soils consisting of clean sand or fine gravel, or a mixture of both 4.00

The net duty may be estimated as follows: For a probable yield of 10 tons of alfalfa per acre, the soil being sandy with a good percentage of clay, we have since one ton requires 5 ins. or 0.416 acre-feet, 0.417 × 10 × 1.5 = 6.25 acre-feet.

The amount required for a probable yield of 30 bus. of wheat is found as follows: 30 bus. of wheat = 30 × 60 = 1,800 lbs. The average relative weights of kernels and straw are as 1 to 1.5. Hence 30 bus. of wheat represent 1,800 × 2.5 = 4,500 lbs. = 2.25 tons. This gives for the quantity required, in case of a light sandy soil 0.415 × 2.5 × 2 = 2.08 acre-feet.

From these quantities it is necessary to deduct a percentage of the minimum amount of rain falling within the period of the activities of the plant. On the Pacific coast this period includes, for alfalfa, the time between March 1 and October 1, or 7 months. On the middle Pacific coast the minimum rainfall during this period is 1.3 ins. Deducting from this 25 per cent as due to showers too light to be effective, there remain 0.975 ins. = 0.08 acre-foot. Hence the probable amount to be supplied to raise 10 tons of alfalfa on the middle Pacific coast will be 6.25 − 0.08 = 6.17 acre-feet.

In case wheat is grown in the same section, the rainfall to be considered practically includes the fall from seed to harvest time, including the months from January to the first of June, which is equal to 12.3 ins. Deducting from this, as above, 25 per cent as due to ineffective rains there remain 9.225 ins. or 0.769 acre-feet. The probable amount to be supplied to produce 30 bus. of wheat in a light sandy soil in the section described in a dry year is consequently 2.08 − 0.769 = 1.311 acre-feet.

The minimum amount of rain falling within the period of the activities of the plant is taken in consideration in order to insure the growing of a crop in every season. Fully 50 per cent of the amount of 1.311 acre-feet above may be considered to be an insurance against a very dry season.

By the method outlined above it is possible to estimate the amount of water to be supplied for any probable yield for the different classes of soils and different seasons with a fair degree of accuracy. The minimum amounts of rainfall in the different sections are found in Table X.

PERCOLATION.

In districts where the art of irrigation has found its highest development, cement pipes or cement lined covered conduits of other descriptions convey the quantities of water delivered by pumps, drawn from storage reservoirs or diverted from streams, undiminished to the points of application. In sparsely settled districts, where the value of water is not sufficiently high to call for its total conversation, unlined canals and ditches prevail. The losses due to percolation in unlined channels are sometimes enormous. According to Saville (U. S. Dept. of Agr.) the losses vary between 0.23 and 6.1 cu. ft. per day for every square foot of the wet surface of the channel. The seepage losses per mile of the channel have been found to vary between 0.19 and 5.73 per cent of the quantities carried. Losses of 12 to 15 per cent per mile are found where the channels pass through very porous, sandy or gravely soils as in the case of the Tule canal in California (15.73 per cent per mile) and the Umatilla canal in Oregon (12 per cent per mile).

TABLE VI. ACRE-FEET PER SEASON, RAINFALL NOT INCLUDED.

Description of crop	Arizona.	California.	Idaho	Montana	Texas.	Utah	Wash	Wyom
Alfalfa		1.50	3.10	1.33	[illegible]	3.71	[illegible]	.
Barley	1.60			1.41	.		.	[illegible]
Corn	2.10				[illegible]			0.7[illegible]
Oats			[illegible]	1.71		1.75		1.50
Onions	6.20				2.10			...
Trees, deciduous	1.27	2.00	2.11	1.13		[illegible]	[illegible]	
Trees, citrus		1.9[illegible]						.
Peas	[illegible]			.73				
Potatoes	2.10							.6[illegible]
Rice					[illegible]			
Sugar beets	.50			1.40				
Tomatoes	1.70							
Wheat	[illegible]			1.88		1.4[illegible]		
Hops						.43		
Old meadow			2.38					
New meadow								
Strawberries	6.20							

In the majority of cases investigated by Saville the daily losses per square foot of wetted area were found to vary between 0.5 and 1.5 cu. ft.

The losses due to percolation depend: (1) on the transmission capacity of the soil which varies with the porosity, the effective diameter of the soil-grains and the temperature of the water.

(2) On the elevation of the plane of saturation in a vertical passing through the channel. When the plane of saturation is on a level with the surface of the water in the channel there is a gain instead of a loss, provided the water in the channel is in motion: seepage begins with the fall of the watertable below the bed of the channel and increases in amount with increasing depression of the watertable.

(3) The depth of water in the channel. The quantities percolating increase with the effective head which includes the depth of water in the channel and the vertical distance traversed; they decrease with the distance of percolation. The influence of the depth of water decreases rapidly with increasing distance of percolation and finally vanishes. On this account the losses are greatest when water is first turned into a channel, they diminish rapidly and finally become constant.

TABLE VIII.

Values of S in Formula Q=KSD. Side slope of channel.

Mean width of channel.	0.5:1.	1:1	1.5:1.	2:1.	2.5:1	3:1.
2D	0.228	0.234	0.251	0.273	.....	.....
3D	0.289	0.295	0.312	0.335	0.356	0.378
4D	0.350	0.356	0.374	0.395	0.422	0.448
5D	0.411	0.417	0.435	0.456	0.483	0.509
6D	0.172	0.478	0.496	0.517	0.554	0.570
8D	0.591	0.600	0.618	0.639	0.666	0.692
10D	0.717	0.722	0.740	0.761	0.788	0.811
12D	0.839	0.844	0.862	0.883	0.910	0.936
14D	0.961	0.966	0.986	1.007	1.032	1.058
16D	1.083	1.088	1.106	1.129	1.154	1.180
18D	1.205	1.210	1.228	1.251	1.276	1.302
20D	1.327	1.333	1.350	1.373	1.298	1.424

(4) The length of time a channel has been in use. If the plane of saturation is below the bed of the channel, percolation will cause it to rise slowly to higher levels and the percolation losses will gradually decrease with the rise of the watertable.

In calculating the losses from unlined canals, the loss due to evaporation from the surface is usually neglected and the basis is the seepage loss per square foot of the wet surface per day. This may be taken to be equal, excluding extremes, to

0.2-0.3 for channels in heavy clay soils, or for old channels coated with silt.
0.3-0.5 for channels in common clay soils.
0.6-0.8 for channels in clayey loams and loamy clays.
1.0-1.15 for channels in sandy loams.
1.5-1.6 for channels in light loamy sands.
6.0-6.5 for channels in clean sand or sand with gravel.

The porosity of the concretes when used as ditch linings depends upon their composition, those rich in cement being the least porous. Their effectiveness in preventing percolation depends further on the thickness of the slab, the head of water above it and on the material in the back fill. A 6-in. slab is twice as effective as a 3-in. one; a depth of 2 ft. of water causes twice the flow due to a depth of 1 ft. A lining consisting of cobble stones laid in cement mortar is practically impervious; a 6-in. concrete slab rich in cement nearly so. A simple 1-in coat of cement mortar on a smoothed soil surface is effective when new; in channels not always in use its durability is, however, short; it will soon crack and peel off. A coat of heavy asphaltic oils is somewhat less effective but more durable.

TABLE IX.

State	Stream.	Canal.	Acre-feet. Canal.	Acre-feet. Lateral.	Percentage for lateral.
Arizona	Salt River	Several	3.42	....	..
California	Santa Ana	Gage	2.16	....	..
California	Santa Clara	Several	2.00	....	..
California	Tule River	Pioneer	8.01	1.41	18
California	Tuolumne	Modesto	13.18	5.76	44
California	Tuolumne	Turlock	8.34	7.69	92
Colorado	Arkansas	Amity	4.92	1.82	47
New Mexico	Pecos	Pecos	7.90	3.69	38
Utah	Bear River	Bear River	1.84	1.81	44
Washington	Yakima	Sunnyside	6.25	4.73	75

The values of K (the loss per square foot per day) given below are based on experiments made by Adams and Etcheverry, University of California. The water in the experimental channel had a depth of 3.5 ft. and the value of K for the back fill is assumed to be 0.5

Nature of lining.	Value of K.
Cement concrete, 3 ins. thick	0.066
Cement-lime concrete, 3 ins. thick	0.172
Coat of cement mortar	0.173
Coat of heavy oil, 3.67 gals. per sq. yd.	0.248
Clay puddle, 3.5 inches thick	0.261
Coat of heavy oil, 3 gals. per sq. yd	0.310
Coat of heavy oil, 2.33 gals. per sq. yd	0.363
Coat of thin oil, 2.5 gals. per sq. yd	0.463

Let

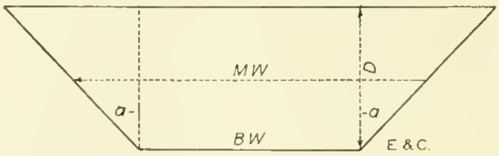
BW = the bottom width of a channel,
D = its depth,
a = the horizontal projection of the side walls per foot of rise.
= tang a,

then for the trapezoid

$$BW = \text{Mean Width} - Da$$

and the wet circumference of the trapezoid

$$= MW - Da + 2\sqrt{D^2 + (aD^2)}$$



Let

K = the seepage loss per square foot per day,
Q = the loss in second-feet per mile of channel.

then for the trapezoid

$$Q = \frac{K[MW - Da + 2\sqrt{D^2 + (aD^2)}]5280}{86400}$$

$$= K[MW - Da + 2\sqrt{D^2 + (aD^2)}]0.0611$$

and the percentage of loss per mile, V being the velocity in feet per sec.

$$\% \quad \frac{K[MW - Da + 2\sqrt{D^2 + (aD^2)}]0.0611\text{x}100}{MW\text{x}D\text{x}V}$$

Let

$MW = 24$
$D = 8$
$a = 2$
$K = 0.5$

then

$$Q = 0.5(24 - 8\text{x}2 + 2\sqrt{8^2 + (2\text{x}8)^2})0.0611$$
$$= 1.338 \text{ sec.-ft.}$$

The percentage of loss, for $V = 2$

$$\frac{1.338 \times 100}{24 \times 8 \times 2} = 0.348 \text{ per cent.}$$

Hanna gives the following losses (Table VII) in per cent of flow per mile of channel.

TABLE VII.

Capacity of channel in second-feet.	Loss in per cent of flow per mile. K=0.5.	K=1.0.	K=1.5.
10 or less	1	8	12
11-25	2.5	4.5	7
26-50	1.5	3	4.5
51-95	1	2	3
76-100	0.75	1.5	2.5

The formula given above may be simplified. To find the power of the cross-section of the channel to which the seepage loss is proportional we may put

$$X = \frac{\log\ (\text{Loss a}) - \log\ (\text{Loss b})}{\log\ (\text{Section a}) - \log\ (\text{Section b})}$$

which gives

$$X = 0.5$$

Consequently the losses are proportional to $\sqrt{\text{area of section}}$. The area of the section may be conceived to be composed of a number of squares, each of a depth D, so that

$$\text{area} = nD^2$$

and, $n = \dfrac{\text{Mean Width}}{D}$ = the relative width of the channel.

Consequently, $\sqrt{\text{area}} = \sqrt{\text{relative width}} \times D$.

Making this substitution for the area of the section, the equation for the seepage loss per mile of channel may be written

$$Q = KSD$$

in which K has the same significance as above and S is a coefficient varying with the relative width of the channel $\dfrac{MW}{D}$ and the side slope tang a. Values of S are found in Table VIII.

Example:

Let $K = 0.5$, $D = 10$, relative width = 4, tan $a = 3$.

From the table, for a mean width = $4D$ and tan $a = 3$, S = 0.448.

Inserting values

$$Q = 0.5 \times 0.448 \times 10 = 2.24 \text{ sec.-ft. per mile.}$$

The percentage of loss, for $V = 2$.

$$\frac{0.5 \times 0.448 \times 10 \times 100}{40 \times 10 \times 2} = 0.28 \text{ per cent per mile.}$$

THE GROSS DUTY.

The gross duty of water denotes the quantity required to irrigate a unit area of land per season plus the losses due to seepage and evaporation from unlined canals and ditches.

Table IX, published by the U. S. Department of Agriculture, gives the quantities in acre-feet carried by main canals and the laterals diverted therefrom.

METHOD OF ESTIMATING THE GROSS DUTY OF WATER.

In discussing the net duty of water we have seen that the quantity required for a probable yield of 10 tons of dry matter per acre on the middle Pacific coast is 6.17 acre-feet. In the interior valleys of this section, five crops of alfalfa mature during a season and since each crop requires two irrigations, water will be applied ten times during the season. In case the field contains one square mile or 640 acres, one irrigation will probably last six days or 60 hours and the ten irrigations 600 hours or 2,160,000 seconds. A depth of 6.17 ft. on 640 acres will be equal to 172,000,000 cu. ft., which divided by the number of seconds is equal to 79.64 sec.-ft.

Let the length of the distributing channel be four miles, the velocity of flow 1.66 ft., the sideslope of the channel 2:1, its mean width 12 and its depth 4 ft. and the probable seepage losses will be, since $S = 0.335$ according to Table X, and assuming $K = 1.5$.

$$Q = 1.5 \times 0.335 \times 4 \times 4 = 8.04 \text{ sec.-ft.}$$

Consequently the quantity diverted into the lateral must be 79.64 8.04 = 87.68 sec.-ft.

Let there be 10 square miles under a canal, each requiring an equal amount of 87.68 sec.-ft. Let the canal be 20 miles long, the velocity of flow 2 ft. per second, the sideslope 2:1, the mean width = $5D$, the depth $\sqrt{\dfrac{\text{area}}{5}} = 9.36$ ft. Then, since $S = 0.456$ and assuming $K = 1.5$

$$Q = 1.5 \times 0.456 \times 9.36 \times 20 = 128 \text{ sec.-ft.}$$

Since the quantity carried by the laterals is 876.8 sec.-ft., the amount to be diverted into the main canal must be 876.8 + 128 = 1004.8 sec.-ft. Dividing the total by the number of square miles gives 100.48 sec.-ft. per square mile and the gross duty in acre-feet will be per acre

$$\frac{100.48 \times 6.17}{79.64} = 7.785 \text{ acre-feet.}$$

TABLE X.—MINIMUM MONTHLY PRECIPITATION IN WESTERN REGIONS.

District.	Jan.	Feb.	Mar.	April.	May.	June.	July.	Aug.	Sept.	Oct.	Nov.	Dec.
North Dakota	0.6	0.5	0.5	1.4	2.1	3.5	2.1	1.1	0.8	1.0	0.5	0.6
Upper Mississippi	0.8	0.9	1.5	2.2	3.3	3.8	2.7	2.0	2.5	1.8	0.7	1.2
Missouri Valley	0.5	0.3	0.8	1.9	2.4	3.4	2.2	1.7	1.0	0.7	0.5	0.5
Northern Slope	0.6	0.4	0.5	1.0	1.6	1.2	0.8	0.8	0.6	0.7	0.3	0.3
Middle Slope	0.4	0.5	0.5	1.2	1.8	1.2	1.7	1.5	0.4	0.7	0.3	0.5
Southern Slope	0.6	1.2	0.6	0.9	2.0	3.2	1.7	2.6	2.1	1.4	0.3	0.7
Southern Plateau	0.4	0.4	0.3	0.1	0.0	0.0	0.1	0.2	0.0	0.3	0.2	0.5
Middle Plateau	1.1	0.9	0.8	0.8	0.6	0.4	0.1	0.1	0.3	0.4	0.1	1.2
Northern Plateau	1.6	1.3	1.4	1.2	1.1	1.1	0.3	0.2	0.6	1.1	0.9	1.6
North Pacific Coast	5.3	3.0	2.2	2.0	1.3	1.2	0.4	0.2	1.0	2.7	3.7	5.5
Middle Pacific Coast	3.8	3.2	2.9	1.7	0.7	0.1	0.0	0.0	0.3	0.9	2.1	3.7
Southern Pacific Coast	1.3	1.1	1.3	0.7	0.3	0.0	0.0	0.0	0.1	0.4	0.8	1.5

In the application of Table VIII it is well to emember that

for Mean Width	the Mean Width	the Depth
$=2D$	$\sqrt{2\,\text{area}}$	$\sqrt{\dfrac{\text{area}}{2}}$
$=3D$	$\sqrt{3\,\text{area}}$	$\sqrt{\dfrac{\text{area}}{3}}$
$=4D$	$\sqrt{4\,\text{area}}$	$\sqrt{\dfrac{\text{area}}{3}}$

etc., etc.

Methods of Finding Center Cut for Economic Locations of Banked Irrigation Ditches.

Contributed by F. C. Snow, Assistant Professor of Civil Engineering, Montana State College, Bozeman, Mont.

In laying out irrigation ditches with banks on both sides or with a bank on the lower side of the ditch only, if the location is to be economic; that is, if the amount of excavation is to equal the amount of embankment, or the amount of excavation is to equal the amount of embankment plus shrinkage and waste, it is necessary to know what cut is required at the center of the ditch at each station so that the above conditions may be fulfilled.

An easy way of obtaining this center cut is by means of a diagram having for a basis the slopes of the surface at right angles to the center line of the ditch and for ordinates the proper center cuts. Following are methods for constructing diagrams of this kind for canals having any desired width of bottom, any top of banks, any depth, any slopes of sides of ditch and banks and any percent allowance for waste and shrinkage, with banks on both sides of the ditch or a bank on the lower side of the ditch only. They are based on formulas derived by equating an expression for area in excavation times (one minus the percent of shrinkage) to a similar expression for the area in embankment.

NOTATION.

b_c = width of bottom of ditch in feet (base in cut).

b_f = width of top of banks in feet (base in fill).

h = the distance in feet from the top of the banks to the bottom of the ditch measured vertically (depth of ditch).

s = slope of the surface on a line at right angles to the center line of the canal, equals tan. Alpha in Fig. 2 equals 0.05 for a rise of 0.5 ft. vertically in 10.0 ft. horizontal distance.

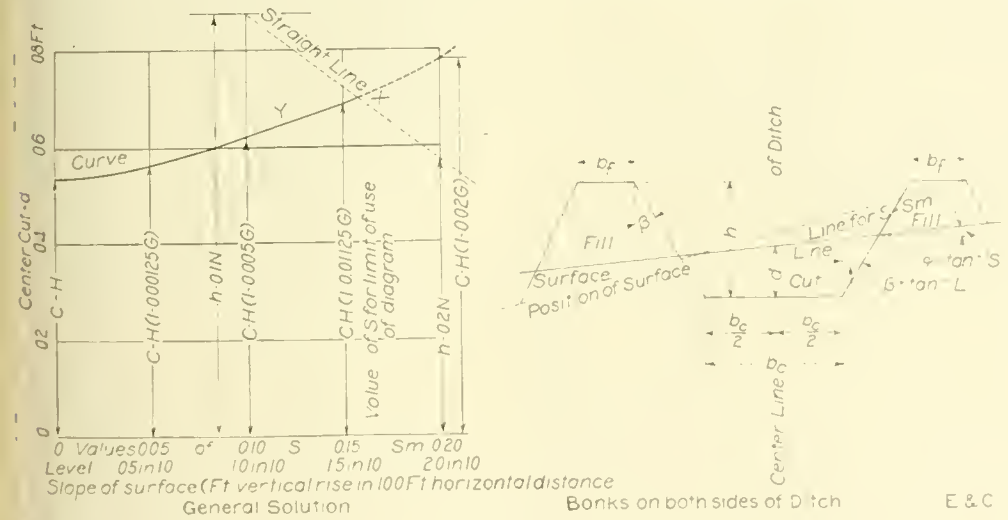
L = slope of sides of banks and ditch, equals tan. Beta in Fig. 2 equals 1.5 for a side slope of 1.5 horizontally to 1 vertically.

p = per cent of decrease of the volume of earth in the banks from its volume in the ditch, expressed as a decimal, equals 0.1 if the shrinkage allowance is to be 10% or 0.9 times excavation equals embankment.

d = center cut in ditch so that excavation equals embankment or so that excavation equals embankment plus shrinkage and waste.

CASE 1. BANKS ON BOTH SIDES OF THE DITCH.

Figure 1 shows a general method for constructing a diagram for this case. The values of the various letters are given below.



General Solution

Banks on both sides of Ditch

Fig. 1.

(A) Where excavation equals embankment plus shrinkage and waste:

$$C = \frac{(1-p)\dfrac{b_c}{2} + 2Lh + b_f}{(1+p)L}$$

$$D = \frac{2h(b_f + Lh)}{(1+p)L}$$

$$H = \sqrt{C^2 - D}$$

$$N = \frac{b_c}{2} + Lh + b_f$$

$$G = \frac{1+p}{1-p}L$$

(B) Where excavation equals embankment:

$$C = \frac{\dfrac{b_c}{2} + b_f}{L} + 2h$$

$$D = 2h\left(\frac{b_f}{L} + h\right)$$

$$H = \sqrt{C^2 - D}$$

$$N = \frac{b_c}{2} + Lh + b_f$$

$$G = L$$

TABLE I.—FOR PLOTTING DIAGRAM FOR CASE 1, FIG. 1.

For Plotting Curve Y.

Surface slope	Value of s.	Value of center cut.
Level	0	C—H
0.5 in 10.0	0.05	C—H(1-0.00125 G)
1.0 in 10.0	0.10	C—H(1-0.005 G)
1.5 in 10.0	0.15	C—H(1-0.01125 G)
2.0 in 10.0	0.20	C—H(1-0.02 G)

For Plotting Straight Line X.

Surface slope.	Value of s.	Value of center cut.
1.0 in 10.0	0.10	h—0.1 N
2.0 in 10.0	0.20	h—0.2 N

The intersection of the line X and the curve Y determines a value of s for the limit of the use of this diagram, which value of s is called s_m and is the slope of the surface when the line representing the surface passes through the upper outside corner of one of the banks as shown by the dotted line in Fig. 2; s_m is therefore the largest value of s for which it is possible to use the diagram.

CASE 2. A BANK ON THE LOWER SIDE OF THE DITCH ONLY.

Figure 2 shows a general method of constructing a diagram for this case. The values of the various letters are given below.

(A) Where excavation equals embankment plus shrinkage and waste:

$$K = I - \sqrt{I^2 - J}$$

$$I = \frac{(1-p)\,b_c + 2Lh + b_f}{2pL}$$

$$J = \frac{h\,(b_f + Lh)}{pL}$$

$$M = \frac{2hL + b_f - \dfrac{pb_c}{4}}{2 - \dfrac{p}{2}}$$

(B) Where excavation equals embankment:

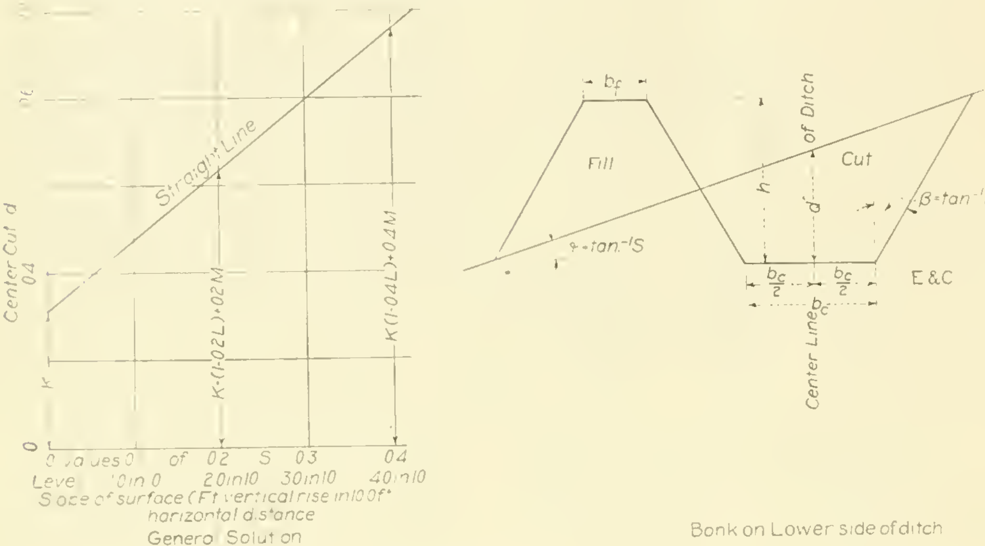
$$K = \frac{h\,(b_f + Lh)}{b_c + 2Lh + b_f}$$

$$M = \frac{b_f}{2} + Lh$$

TABLE II. FOR PLOTTING THE STRAIGHT LINE OF DIAGRAM—CASE 2, FIG. 2.

Surface slope	Value of s.	Value of center cut.
level.	0	K
2.0 in 10.0	0.20	K(1-0.2L)+0.2M
4.0 in 10.0	0.40	K(1-0.4L)+0.4M

After a diagram has been drawn by one of the methods explained above the position of the center stakes for an economic location can be determined very easily. For example, b_c equals 5 ft., b_f equals 3 ft., h equals 3 ft., L equals 1.5, p equals 0.1 and there are to be banks on both sides of the ditch. Figure 3 shows a diagram for these values for all cases listed above and the computations for the same are given below. Suppose that the ditch is to be located on a 0.01 per cent grade with an elevation at the bottom at Sta. 0 of 100 and the surface slope at this station as determined by finding the difference in elevation between two points 10 ft. apart on a line at right angles to the center line of the canal is 1.0 in 10.0, then the diagram shows that there should be a cut of 1.86 ft. at this station. 100 plus 1.86 equals 101.86, which is the elevation of the surface at Station 0 on the center line of the ditch. A lath or other tall stake is set up at a point on the ground having this elevation. At 100 ft. from this point, approximately along the line of the ditch a trial point for Station 1 is assumed and the surface slope is determined as before, say that it equals 0.3 in 10.0, the center cut from the diagram corresponding to this surface slope is 1.77 ft. The elevation of the bottom of the ditch is 100 minus 0.01 equals 99.96 and of the surface is 99.96 plus 1.77 equals 101.73. A point is now found which has this elevation and is 100 ft. from Station 0. If this point is fairly close to the trial point it can be taken as Station 1, but if the two points are some distance apart so that the slope of the surface is not the same as it was for the trial point then a second trial is made. When Station 1 has been finally located by this method a tall stake or lath is set up. Station 2 is located in the same manner 100 ft. from Station 1. Station 3, 100 ft. from Station 2 and so on throughout the length of the canal. Each station being 100 ft. from the preceding station and at a point having the proper surface elevation and tall stakes are set at all stations.



General Solution

Bank on Lower side of ditch

Fig. 2.

Computations for Diagrams. Fig. 3.

Case 1-A. Computations for Curve 1, Fig. 3.

Example:—$bc = 5$, $bt = 3$, $h = 3$, $L = 1.5$, $p = 0.1$.

$$C = \frac{0.9 \times \frac{5}{2} + 2 \times 1.5 \times 3 + 3}{1.1 \times 1.5} = \frac{14.25}{1.65} = 8.63.$$

$$D = \frac{2 \times 3\,(3 + 1.5 \times 3)}{1.1 \times 1.5} = \frac{45}{1.65} = 27.27.$$

$$N = \frac{5}{2} + 1.5 \times 3 + 3 = 10.$$

$$H = \sqrt{8.63^2 - 27.27} = \sqrt{47.21} = 6.27.$$

$$G = \frac{1.1}{0.9} \times 1.5^2 = 2.75.$$

For plotting Curve Y.

$s = 0$, $d = 8.63 - 6.87 = 1.76.$
$s = 0.05$, $d = 8.63 - 6.87\,(1 - 0.00125 \times 2.75) = 8.63 - 6.85 = 1.78.$
$s = 0.10$, $d = 8.63 - 6.87\,(1 - 0.005 \times 2.75) = 8.63 - 6.77 = 1.86.$
$s = 0.15$, $d = 8.63 - 6.87\,(1 - 0.01125 \times 2.75) = 8.63 - 6.66 = 1.97.$

For plotting straight line X.

$s = 0.10$, $d = 3 - 0.1 \times 10 = 2.0.$
$s = 0.20$, $d = 3 - 0.2 \times 10 = 1.0.$

Case 1-B. Computations for Curve 2, Fig. 3.

Example:—$bc = 5$, $bt = 3$, $h = 3$, $L = 1.5$, $p = 0$.

$$C = \frac{\frac{5}{2} + 3}{1.5} + 2 \times 3 = 9.67$$

$$D = 2 \times 3 \left(\frac{3}{1.5} + 3\right) = 30.0.$$

$$N = \frac{5}{2} + 1.5 \times 3 + 3 = 10.$$

$$H = \sqrt{9.67^2 - 30} = \sqrt{63.51} = 7.97$$

$$G = 1.5^2 = 2.25$$

For plotting Curve Y

$s = 0$, $d = 9.67 - 7.97 = 1.70.$
$s = 0.05$, $d = 9.67 - 7.97\,(1 - 0.00125 \times 2.25) = 9.67 - 7.95 = 1.72$
$s = 0.10$, $d = 9.67 - 7.97\,(1 - 0.005 \times 2.25) = 9.67 - 7.87 = 1.80$
$s = 0.15$, $d = 9.67 - 7.97\,(1 - 0.1125) = 9.67 - 7.77 = 1.90.$

For plotting straight line X

$s = 0.10$, $d = 3 - 0.1 \times 10 = 2.0.$
$s = 0.20$, $d = 3 - 0.2 \times 10 = 1.0.$

Case 2-A. Computations for Curve 3, Fig. 3.

Example:—$bc = 5$, $bt = 3$, $h = 3$, $L = 1.5$, $p = 0.1$

$$I = \frac{0.9 \times 5 + 2 \times 1.5 \times 3 + 3}{2 \times 0.1 \times 1.5} = \frac{16.5}{0.3} = 55.$$

$$\frac{3\,(3 + 1.5 \times 3)}{0.1 \times 1.5} = \frac{22.5}{0.15} = 150.$$

$$K = 55 - \sqrt{55^2 - 150} = 55 - 53.62 = 1.38.$$

$$M = \frac{2 \times 3 \times 1.5 + 3 - \frac{0.1 \times 5}{4}}{2 - \frac{0.1}{2}} = \frac{11.87}{1.95} = 6.08.$$

For plotting straight line.

$s = 0$, $d = 1.38.$
$s = 0.40$, $d = 1.30\,(1 - 0.40 \times 1.5) + 0.4 \times 6.08 = 2.99.$

Case 2-B. Computations for Curve 4, Fig. 3.

Example:—$bc = 5$, $bt = 3$, $h = 3$, $L = 1.5$, $p = 0$.

$$K = \frac{3\,(3 + 1.5 \times 3)}{5 + 2 \times 1.5 \times 3 + 3} = \frac{22.5}{17} = 1.32.$$

$$M = \frac{3}{2} + 1.5 \times 3 = 6.0.$$

For plotting straight line.

$s = 0$, $d = 1.32.$
$s = 0.40$, $d = 1.32\,(1 - 0.4 \times 1.5) + 0.4 \times 6.0 = 2.93.$

If the canal center should follow exactly the line of stakes located as above the location would be economic. However, a final location of the line that averages the position of these stakes and is made up of a series of tangents and curves will not materially change the amount of excavation and embankment and will give a much neater and in general a shorter ditch. The tall stakes set at the stations aid greatly in establishing this final location as a number of them can be seen at one time and an average line easily located on the ground without plotting. If desired, however, a traverse may be run locating the positions of these preliminary stakes from which the line can be plotted and the final location made on this plot and transferred to the ground. This process is almost necessary when working in brush or woods.

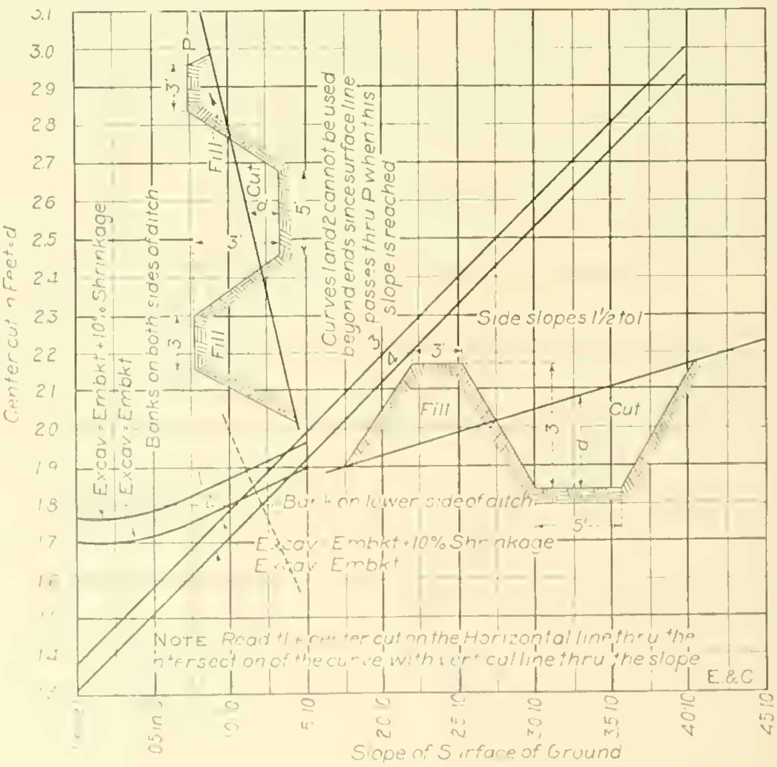


Fig. 3.

After the final line is located stakes are set at 100 ft. stations on it, levels are run over it, the grade elevations determined and the slope stakes set by the same methods as used in other earth work surveys.

Surveys and Plans for a Comprehensive Land Reclamation and Drainage Project.

(Staff Article.)

The land reclamation and drainage project, plans for which are presented in this article, is exceptional. A list of the works involved in land reclamation and drainage includes principally levees, pumping works, channel enlargement, open ditches, tile drains, sluices, detention basins, settling basins, and outlets. Few specific projects require all of these structures and for a still smaller number are they all included and provided for in the original reclamation plans. This is one reason why the drainage and reclamation works of very many reclaimed areas are not in ideal coordination and proportion; they have been added one after another to an original partial plan. The exceptional feature of the project illustrated is that it includes nearly all of the list of works belonging to land reclamation and drainage and that a coordinated plan providing for all has been worked out.

General Plan.—The area being reclaimed is shown by the map Fig. 1 and is known as the Marion County Drainage District, being situated in Marion county in Missouri. This area is bounded on the east by the Mississippi River, on the north and south by small rivers which flow into the Mississippi and on the west by a ridge of highland. All principal topography is indicated on the map Fig. 1, on which are also shown the reclamation works These embrace:

(1) The prevention by levees of overflow from South Fabius River, the Mississippi River and North River.

(2) The construction of main and lateral ditches and tile drains and outlets for the drainage of the flatlands within the district. The construction of a holding or detention basin in a part of the Eastern Slough by cleaning out all drift and making the cross-section uniform.

(3) The construction of a pumping plant with pump house and machinery, sluiceway, suction and discharge bays, and other necessary appurtenances to furnish an outlet for waters of the district during all stages of the Fabius, the North and the Mississippi Rivers.

Levees.—The protection of the area from overflow by the bordering streams is accomplished by levees. The location of these levees is shown by Fig. 1 and sections of typical levees are shown by Fig. 2. The levee construction demanded was unusualy exacting; the specifications are therefore of interest and their principal requirements are given as follows:

Base of Levee.—The ground to be occupied by the levee must be cleared of logs, trash, weeds and leaves. Following this, the surface between the slop stakes and to a depth of 6 ins. shall be thoroughly broken by plowing or spading, and all trees, stumps and buried logs removed to a distance of not less than 5 ft. outside of the levee. Whenever by reason of delay, rains or other causes, the surface of the ground upon which the levee embankment is to be built, or whenever any part of the levee work shall have been suspended so that the unfinished surface is compacted and will not form a bond with the new material to be placed thereon, such surface shall be re-broken with plow or spade as above specified.

Exploration Trench.—An exploration trench shall be dug at or near the center line of the levee when the height of the levee is 10 ft. or more, wherever required by the engineer. Said exploration trench shall not be less than 3 ft. deep and 3 ft. wide at the bottom. After the exploration trench is inspected and measured by the engineer, it and all other excavations within the base of the levee shall be filled with approved material. Said material shall be thoroughly compacted while being filled, to the satisfaction of the engineer, if done by scrapers, wagons or appliances other than some form of excavating machine which drops the material from a height of 10 ft. or more. All the foregoing work shall be completed at least 300 ft. in advance of the embankment construction. The side slopes in the exploration trench shall be as nearly vertical as practicable, and shall not exceed 1 ft. horizontal to 1 ft. vertical on each side. The engineer will give instructions for disposing of the excavated material, within or adjacent to the base of the levee. For all of the material which shall be taken from the exploration trench payment shall be made for excavation only. In computing the total amount of earth in the finished levee no allowance will be made for the exploration trench wherever the material excavated from the exploration trench has been deposited within the base of the levee, but if the engineer should require any of the material excavated from the exploration trench to be deposited outside the base of the levee, then an allowance shall be made for the quantity of material so deposited outside the base of the levee, at the contract price for levee.

Construction.—(a) When this preparatory work shall have been done, the embankment construction shall be commenced and unless otherwise authorized by the engineer, will be started full out to the slope stakes and carried regularly up to the gross fill in layers not exceeding 2 ft. in thickness. If the work should be done by steam shovel or other acceptable machinery instead of by teams, the manner of carrying up the fill to be to the approval of the engineer. If dredges are used and the material is so soft that it will not stand to the full height at the slope specified, the work may be

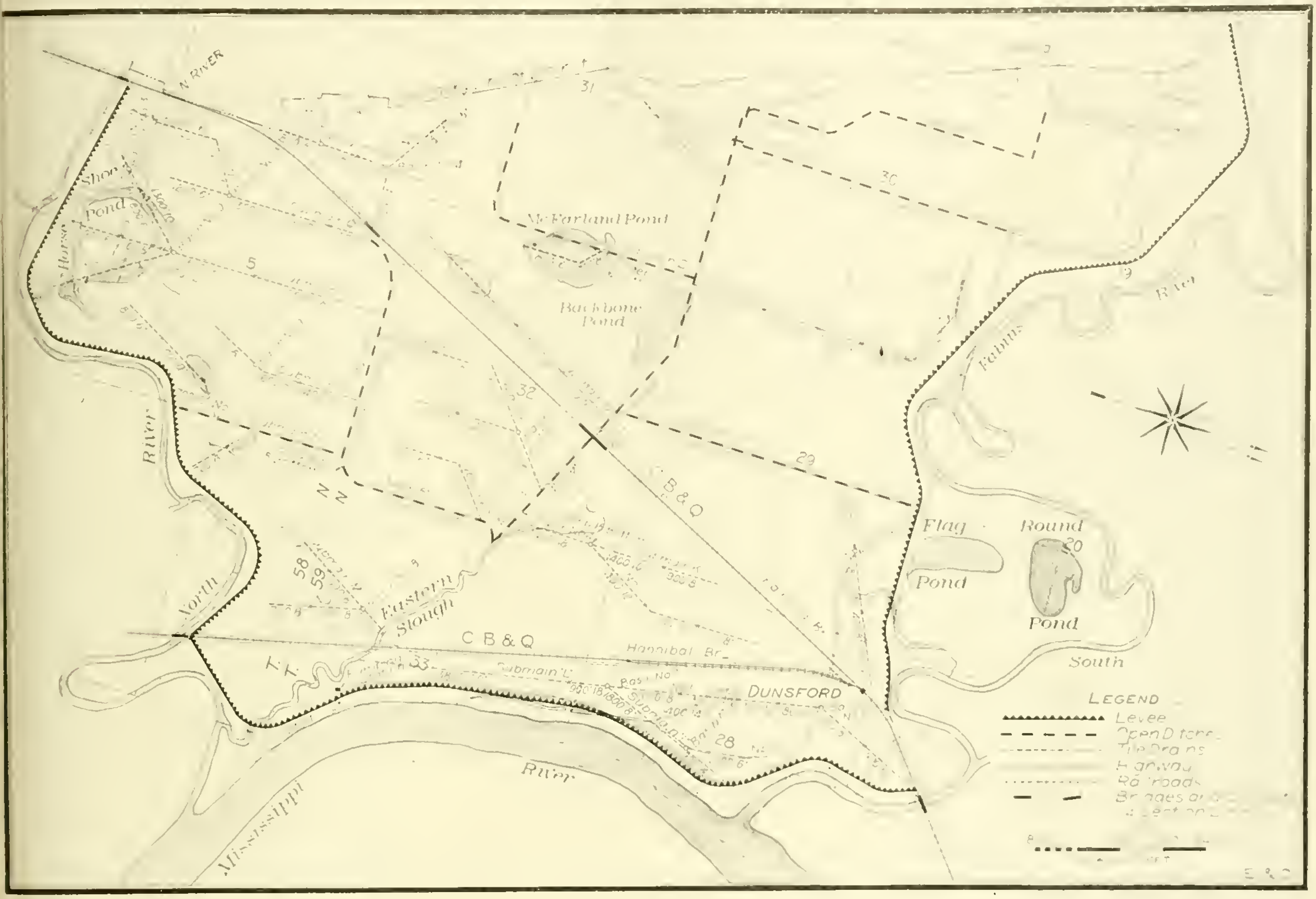


Fig. 1. Map of Comprehensive Drainage System for Marion County, Missouri, Drainage District.

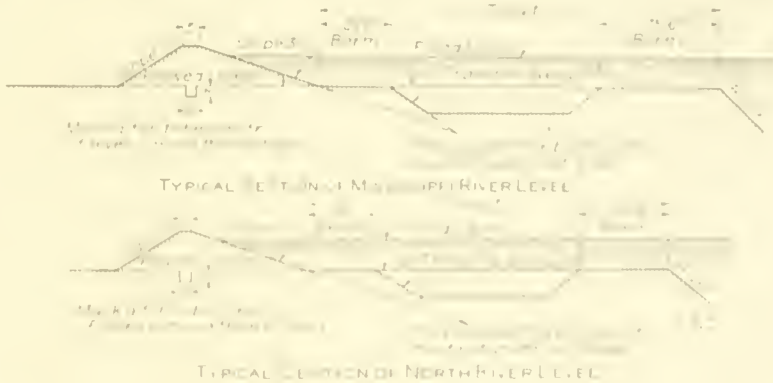

TYPICAL SECTION OF MISSISSIPPI RIVER LEVEE
TYPICAL SECTION OF NORTH RIVER LEVEE

Fig. 2. Typical Levee Sections, Marion County Drainage District.

gone over two or more times to bring the levee to the proper height.

(b) The levee shall be built to such height and width that when fully settled it will be of full height required in the plans and profiles, with top widths and side slopes of the dimensions stated on the profiles. Allowance for shrinkage must be so disposed on top and slopes as to give the required width of crown, and fill out the slopes to plane surfaces from the edge of the crown to base of levee. The allowance required for shrinkage shall be such an amount as the engineer may approve, but in no case shall the amount exceed 25 per cent. Should the contractor fail to provide the allowance required for shrinkage during construction, he will be required to bring the levee to full grade and cross section before acceptance. The contents of the levee computed by the established net grade and slopes only will be paid for.

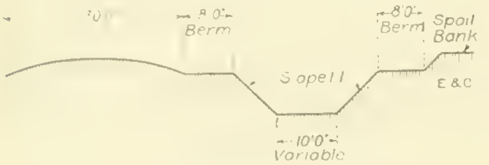


Fig. 3. Typical Ditch Section, Marion County Drainage District.

(c) The alignment of the crown of the levee, when completed, shall not vary from the theoretical location of the crown so as to make abrupt or short bends or reversed curves in the alignment of the crown, or reduce the stability or effectiveness of the levee.

(d) Before crossing any railroad or other permanent crossing, the engineer will inspect that part of the work above same and satisfy himself that the work has been finished and all requirements completed before the contractor shall make said crossing.

(e) The contractor shall give at least 30 days notice to the engineer and the road overseer before he shall cross any public road. After the crossing of any public road, the contractor shall maintain such barriers and red lights at night as shall effectually prevent any accident in consequence of the work done under this contract. These precautions shall be maintained until the highway is restored to its former condition of usefulness and safety.

Sheetpiling.—In any case where there is special danger of settlement or seepage, sheet piling shall be driven to such lengths and dimensions as shall be required by the engineer.

Material Berm and Borrow Pit.—Only clean earth, free from trash and roots, shall be placed in the levee. Such earth shall be procured from the river side of said levee as the engineer may direct. If done by teams, no borrow pit shall be made within a zone of 20 ft. wide outside of and adjoining the base of the levee. If the work is done by drag-line excavator or dredge, a berm 20 ft. wide, unless otherwise specified on the drawings, shall be left at the foot of the levee. No material shall be excavated or removed which would "under-cut" the slope of the levee produced. Where the levee parallels a threatened or cutting bank of the river, the borrow pits shall not be made continuous, but suitable traverses or dams shall be left as directed by the engineer. No frozen material will be allowed to be deposited in the levee and no levee shall be built or extended upon frozen ground.

Open Ditches.—The interior drain system includes an outlet canal, main canals, submains and tile drains. All except the tile drains are open ditch sections of the types shown by Fig. 3. The construction of these ditches was nowise of unusual character.

Tile Drains.—The tile drains system is indicated in Fig. 1, and Figs. 4 and 5 show coordinate structures as mentioned in the abstract of specifications which follows:

Quality.—Drain tile shall be entirely free from cracks and fire checks extending into the pipe in such a way as appreciably to lower its strength. No pipe shall be accepted having pieces broken out in such a way or to such an extent as appreciably to affect the strength of the pipe, or to permit the entrance of soil into the drain. The pipe shells shall have uniform, strong, dense structures throughout, without serious flaws or weak spots.

All pipe shall give a clear ring when stood on end or laid on one side, evenly supported at the lower end or along a line of one side, and free elsewhere, and tapped with a light hammer while dry.

All pipe shall be regular and true in shape. The average diameter shall not be more than 2 per cent less than the specified diameter. No two diameters of the same pipe shall differ from each other more than 7 per cent, nor shall the average diameters of adjacent pipe differ more than 4 per cent.

Pipe may be furnished in lengths of 1, 2, 2½ and 3 ft., but 1 ft. lengths shall not be used for sizes over 15 ins. in diameter. No pipe designed to be straight shall vary from a straight line more than 1½ per cent of its length.

If cement tiles are used, they shall show a uniform dense structure, with clean aggregates, well graded as to size of materials, and with the grains and pieces of aggregate well placed and the pores well filled with good Portland cement. There shall be no spots of specially great porosity. Fractured surfaces shall show broken pieces of aggregate firmly bedded in the concrete. The general appearance of the material shall be at least equal to that of first class gravel concrete in proportions: 1 first class Portland cement; to 2 clean coarse sand; to 1 pebbles, ⅛ in. to one-half thickness of the tile wall in size.

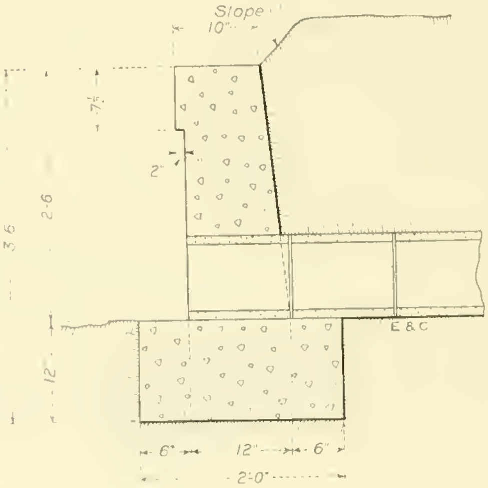


Fig. 4. Outlets for Tile Drains, Marion County Drainage District.

Field Inspection.—The engineer shall very carefully inspect all drain tile as actually delivered along the ditch for use. He will cull out and mark for rejection all poor pipe, and pipe so rejected shall be promptly removed by the contractor.

Digging the Ditches.—The digging of each ditch must begin at its outlet, or at its junction with another tile drain, and proceed toward its upper end. The ditch must be dug along one side of the line of survey stakes, and about 12-ins distant from it in a straight and neat manner, and the top soil thrown on one side of the ditch and the clay on the other. When a change in the direction of the ditch is made it must be done by means of a neat curve, but in all cases the ditch must be kept near enough to the stakes so that they can be used in grading the bottom. In taking out the last draft, the blade of the spade or cutting tool must not go deeper than the proposed grade-line or bed upon which the tiles are to rest.

Grading the Bottom.—The ditch must be dug to the depth indicated by the figures given with the survey, which depth is to be measured from the grade-stakes which are set for that purpose, and graded evenly on the bottom by means of the line and gage method, target, or any other equally accurate device for obtaining an even and true bottom upon which to lay the tile. The bottom must be dressed with the tile hoe, or in case of large tiles with the shovel or cutting tool, in such a way that a groove will be made to receive the tile, so that when laid in it they will remain securely in place.

Laying the Tile.—The laying of the tile must begin at the lower end and proceed up-stream. The tile must be laid as closely as practicable and in lines free from irregular crooks, the pieces being turned about until the upper edges close, unless there is sand or fine silt which is likely to run into the tile, in which case the lower edges must be laid close, and the upper side covered with clay or other suitable material. When, in making turns or by reason of irregular-shaped tile, a crack of ¼ in. or more is necessarily left, it must be securely covered with broken pieces of tile. Junctions with branch lines must be securely and carefully made.

In all tile laying in hard soils, the contractor shall shape the bottom of the ditch approximately to fit the lower 90° (45° each side of the center line) of the circumference of the pipe, taking pains to secure an extra firm bearing near the outer edges of the 90° strip. Upon the concave surface so prepared the contractor shall spread a layer 1 to 2 ins. thick of pulverized soil or sand free from pebbles larger than ¼ in. diameter, and shall firmly bed each pipe truly to line and grade thereon.

Where the bottom of the ditch is so wet and soft as to enable the thorough bedding of the lowest 90° of the pipe without the use of the layer of pulverized earth or sand, and still is firm enough to afford good safe support to the pipe and its load of ditch filling, the engineer may authorize the omission of the layer of granular material, but such authorization shall not excuse imperfections of bedding of the lowest 90° of the pipe circumference.

The space between the pipe and the bottom and sides of the ditch shall be completely packed full by hand with selected earth and tamped with a light tamper as fast as placed all up to the level of the top of the pipe. The side filling shall be carried up as fast on one side of the pipe as on the other.

Binding the Tile.—After the tile have been laid and inspected by the person in charge of the work, they must be covered with clay to a depth of 6 ins. unless, in the judgment of the engineer, the tile are sufficiently firm, so that complete filling of the ditch may be made directly upon the tile. In no case must the tile be covered with sand without other material being first used.

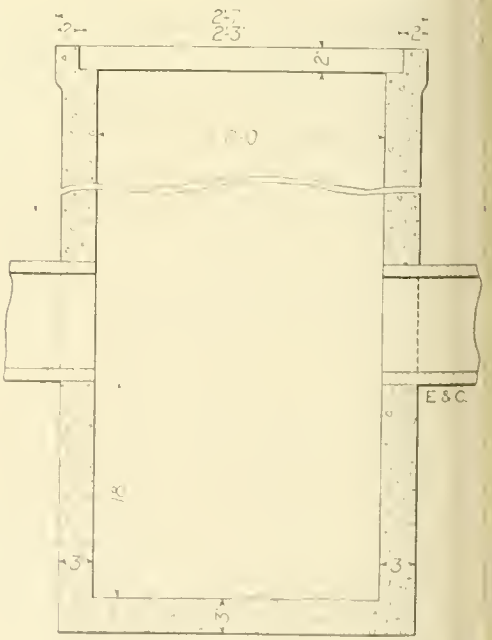


Fig. 5. Settling Basin for Tile Drains, Marion County Drainage District.

Outlet Protection.—Concrete abutments, Fig. 4. shall be constructed at the outlet of all tile drains according to the drawings furnished by the engineer. The concrete shall be made of

1 part Portland cement, 2½ parts of clean sand and 5 parts of crushed stone or gravel, all measured in loose bulk. The pipe used at the outlet shall be vitrified pipe of double strength, hard burned and salt glazed, laid with cemented joints for a distance of 10 ft. back from the point of discharge. The backfilling about the wall shall be thoroughly tamped.

Surface Inlets and Settling Basins.—Combined surface inlets and settling basins shall be constructed as shown by the plans, Fig. 5, and shall be made of concrete consisting of 1 part Portland cement and 7 parts sandy gravel, all measured in loose bulk. The inlets shall be covered with a cast iron grating, made as shown in the plans.

Surface inlets, made by filling the trench with stones 3 ins. to 4 ins. in diameter for a length of 3 to 6 ft., shall be constructed wherever the engineer deems it necessary.

Surface Relief Ditch.—A surface relief ditch shall be constructed at Submain P. This ditch shall be broad and not deeper than 2 ft., and graded to the stakes of the engineer. Said ditch shall be flattened and so constructed that it will offer no obstruction in cultivating the land nor prevent the planting and culture of crops in it.

Pumping Plant.—The pumping station is shown in location on Fig. 1. It comprises a pump, boiler and storage room, with all necessary machinery, and a fuel handling plant. The outlet ditch from the pumping station is 20 ft. bottom width with 1 on 1 sloped banks and 8 ft. beams; it is 11 ft. deep.

Miscellaneous Works.—These include railway and road bridge crossings over canals and a variety of small work.

The engineers for these drainage works were the Edmund T. Perkins Engineering Co., Chicago, Ill.

A Method of Computation for Excavation Tables.

Contributed by Albert S. Fry, Office Engineer, Morgan Engineering Co., Memphis, Tenn.

In Engineering & Contracting for March 11, 1914, is an article by Mr. E. S. Blaine concerning the methods which he used in computing ditch and levee tables adapted to flat country. The Morgan Engineering Co. of Memphis, Tenn., has used tables of a similar character for some time past. The mechanical work involved in the compilation of the tables used by this company is not unlike the operations mentioned by Mr. Blaine. But the underlying theory is somewhat differently conceived. The points of difference may be best brought out by an explanation of the basic principles upon which the tables in use by the Morgan Engineering Co. are founded.

In Figure 1, let D represent the depth of cut, B the width of base, and s the side slope. B and s being constant, the volume of earth in a section of fixed length is a function of the depth D. For a section considered to extend fifty feet each side of a station, this relation is expressed by

$$V=100/27\ (BD+sD^2)\ .\qquad(1)$$

the unit of volume being the cubic yard.

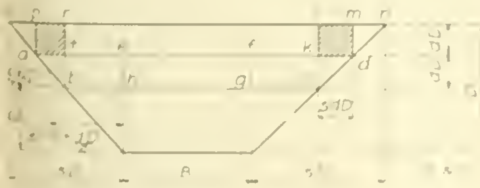


Fig. 1.

Now let us suppose that the depth is increasing uniformly at the rate of 0.1 ft. Then the rate at which the volume is increasing, at any given depth, is found by taking the first differential of (1), which is

$$dV=100/27\ (B+2sD)\ dD\ .\qquad(2)$$

This equation, however, contains the variable D which shows that the volume is not increasing at a uniform rate. We must therefore carry our operations a step further and ascertain the rate at which dV itself is changing. To do this, we again differentiate, the resulting expression being

$$d^2V=100/27\ (2s\times dD^2)\ \ldots\ldots(3).$$

By assigning a fixed value to the increment of depth, dD, it is seen that the equation for d^2V is constant for all values of D. In other words, dV is changing at a uniform rate.

For purposes of illustration, the curves of Fig. 2 were drawn, B and s being considered constant. Equation (1), which is of the second degree, results in a curved line. Equations (2) and (3), which are of the first degree, give straight lines. The dV line shows the rate at which V is increasing for various values of the depth D. The d^2V line shows the rate at which dV is increasing. Since d^2V is constant for all values of D, the d^2V line is parallel to the axis of D, which shows that dV is increasing at a constant rate.

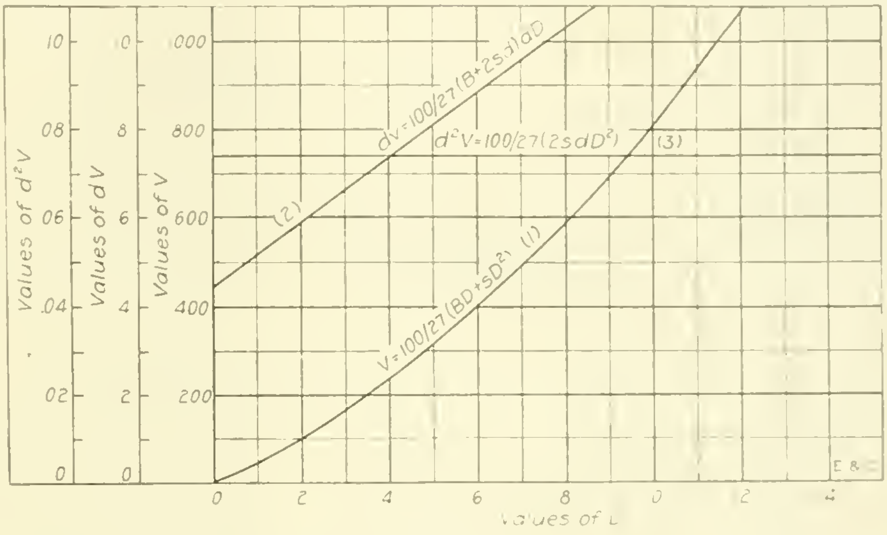


Fig. 2.

Before proceeding further, let us return to Fig. 1 and determine the physical interpretation of each of our differential equations. Let D_1 represent any value of D for which we have computed the corresponding volume V_1. In order to find the volume when D equals D_1+dD, the volume of the prism whose end area is *abcd* must be added to V_1. The area *abcd* is made up of the rectangle *efgh* and the two trapezoids *aehb* and *fdcg*. The mean base of each trapezoid is $s(D_1+\frac{dD}{2})$. The total area *abcd* is then given by $B\times dD+2s(D_1+\frac{dD}{2})dD$, whence

$$dV=100/27(B+2s[D+\frac{dD}{2}])dD\quad(4).$$

This is slightly different in form from (2), the term $\frac{dD}{2}$ not being present in that equation. However (2) is a purely theoretical relation in which D really represents the limit of the expression $(D+\frac{dD}{2})$ as dD approaches zero. But for the purposes at hand, we desire to assign some finite value, such as 0.1 ft., to dD. We have thus taken a step outside of the infinite realm and must therefore not consider the limit of $(D+\frac{dD}{2})$ but $(D+\frac{dD}{2})$ itself. The increment of volume being thus determined, it is only necessary to add the amount dV to V_1 to obtain the volume for D_1+dD.

The volume for D_1+2dD must next be found. To the end area for D_1 we add not only *abcd*, but also *aond* to secure the area for D_1+2dD. The area *aond* exceeds *abcd* by the amount included in the shaded rectangles *aprt* and *lmdk*. The base of each of the latter figures is $s\times dD$ and the height is dD. Therefore their total area equals $2s\times dD^2$ and the corresponding volume is

$$100/27(2s\times dD^2).$$

But this is identical with (3), which shows that the small shaded rectangles represent the second differential, d^2V.

Bearing in mind the fundamental principles heretofore established, it becomes apparent that the most convenient method of building up a set of tables is by a summation process. The work is best performed in two steps which develop from the following relations, an arbitrary value of 1.0 ft. being given to D at the beginning:

For $D=1.0$, $V_{(1.0)}=V_1$ ....(5)

For $D=1.1$, $V_{(1.1)}=V_1+(dV)$ ....(6)

For $D=1.2$, $V_{(1.2)}=V_1+(dV)+(dV+d^2V)$ ....(7)

For $D=1.3$, $V_{(1.3)}=V_1+(dV)+(dV+d^2V)+(dV+d^2V+d^2V)$ ....(8)

For $D=1+n/10$, $V_{(1+n/10)}=V_1+(dV)+(dV+d^2V)$ . . . $+(dV+[n-2]d^2V)+(dV+[n-1]d^2V)$ (9)

The volume for $D=1.1$ is clearly equal to the volume for $D=1.0$ increased by the value of the rate at which the volume is increasing when $D=1.0$. For $D=1.2$, the volume is equal to the volume $D=1.1$, plus the value of the rate at which the volume is increasing when $D=1.1$. But this is equivalent to saying that the volume for $D=1.2$ is equal to the volume for $D=1.1$, plus the value of the rate at which the volume is increasing when $D=1.0$, plus the additional amount by which this rate is itself increasing. Following out this line of reasoning, the general equation (9) is obtained.

In the above equations it will be noticed that the last term in each is the same as the last term in the one immediately preceding it plus the quantity d^2V. Starting with dV, d^2V is added, and a first sub-total obtained; a second d^2V is added and a second sub-total secured. Continuing this process a set of values is found from which the final table is formed. Beginning with the computed value of V_1, to which dV is added, the result being $V_{(1.1)}$. To this sum, the first sub-total is added, giving $V_{(1.2)}$. The second sub-total is then added to $V_{(1.2)}$, $V_{(1.3)}$ being obtained. Proceeding similarly, a table is constructed between any desired limits. For convenience and rapidity of operation, only the first two decimal places are used. A slight cumulative error is thus introduced, for which correction must be made, the interval of such correction depending upon the degree of accuracy desired.

Since this method of calculation is based upon strict mathematical theory, the results obtained are accurate and trustworthy. It is believed that no other method affords as easy and as simple a means for compiling tables of this character. The use of a modern adding machine greatly facilitates the mechanical work involved, making it both rapid and reliable.

Timber Lined Canal New Orleans Drainage System.

(Staff Article.)

A timber lining is being constructed on certain of the canals of the New Orleans drainage system according to plans shown by Fig. 1. As indicated by the general plan the existing canal bottom shown by the dotted line is shaped to provide for the timber lining and from the curbs of this lining the banks are graded by excavation or fill to the new section shown by the full lines. The timber section forms thus a sub-channel carrying all ordinary flow; this timber sub-channel is structurally the special feature of the new canal.

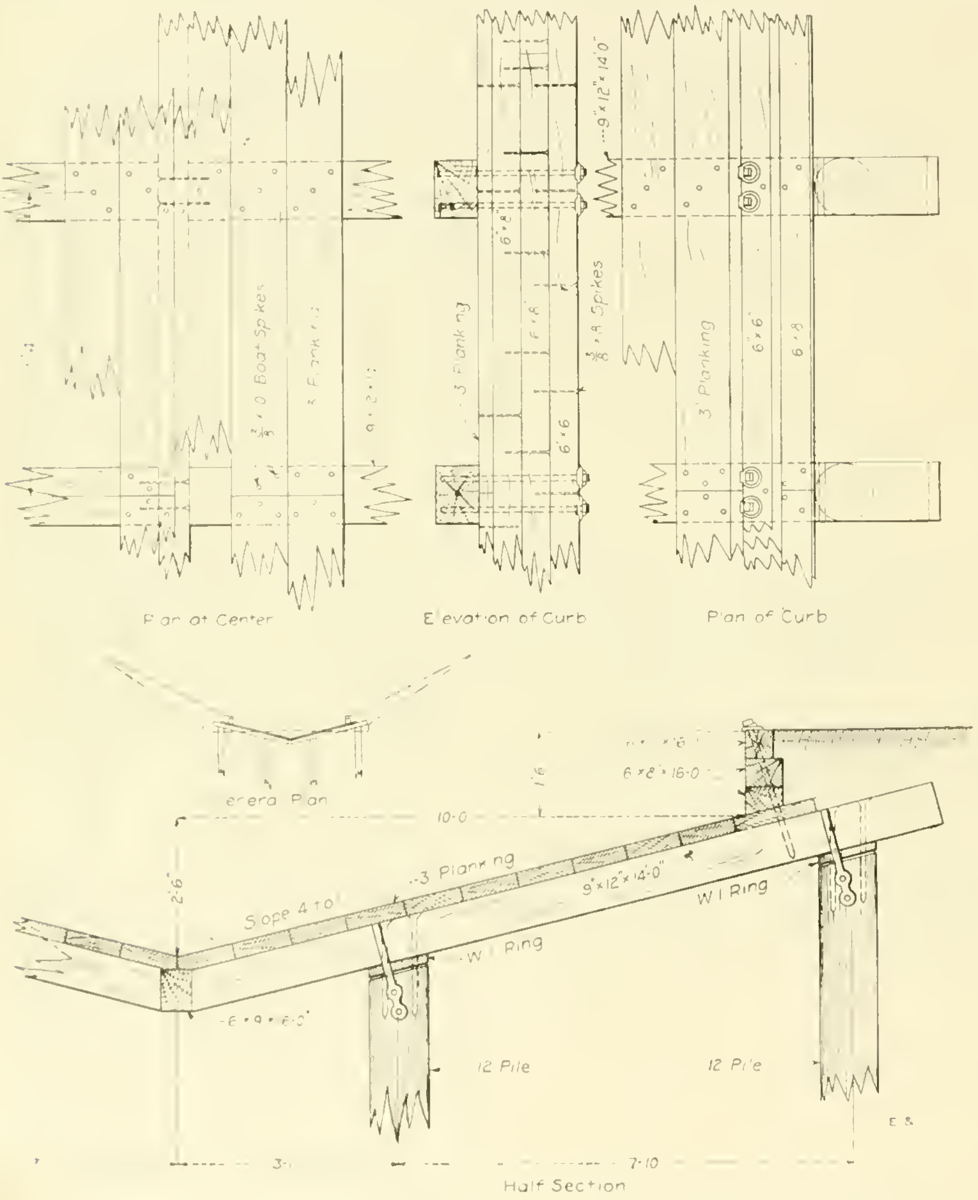


Fig. 1. Diagram Section and Details of Timber Lined Drainage Canal, New Orleans, La.

The general character of the timber lining is shown in the enlarged details. The 12-in. piles range from 25 to 40 ft. in length, are cut to slope on top and are ringed with iron. Piles and transverse caps are of yellow pine and the caps are fastened to the pile heads by both drift bolts and straps. The sheathing for about 6 ft. each side of the center is cypress and for the remaining outside strips is creosoted pine. The sheathing is spiked to the caps with ⅜x8-in. boat spikes, one spike for approximately every 4 ft. B. M. of sheathing. The curb or walling is creosoted pine and is drift bolted through special washers to the cap pieces. All dimensions of timbers are given by the drawings. The design is that of the engineers of the Sewerage-Water Board, New Orleans, La.

Cost of Cement Gun Lining of Small Storage Reservoirs.

To the Editors: I have before me the article on Lining for Small Storage Reservoirs appearing in Engineering & Contracting of March 11, 1914, from which the following is an extract:

> A surface may be also plastered with a cement gun which spreads the plaster evenly by air pressure. It is claimed that this process produces a very impervious plaster with as many as four parts of sand to one part of cement with 10 per cent by volume of the cement of hydrated lime. The cost of this type of lining figures about 15 cts. per square foot for a plaster ½-in. thick. The excessive cost is due to a royalty on a patented machine. Special conditions may justify this high cost however.

As to where the author got his figures on costs of cement gun work of this character or as to whether the particular piece of work on which this figure is based presented unusual difficulties, I am of course uninformed. I desire to say, however, that such a price under ordinary conditions is absolutely unwarranted and a statement such as this which is read by engineers not only on the Pacific Coast, but all over this country, cannot fail to do great damage to the cement gun business. Any concern using cement guns which makes such a quotation is not only committing suicide, but is doing a grievous injustice to those others of us who are doing a legitimate business at legitimate prices and are ignoring the fact that the gun is a patented article.

The General Cement Gun Co. of Chicago is in the business of cement gun contracting in the middle States and I wish to say that on a job of fair proportions, aside from the question of scaffolding, transportation of machines and men and other unknown development expenses and assuming a cost on the job of cement at $1.50 per barrel, sand at $1.50 per cubic yard, labor at an average of 30 cts per hour and gasoline at 20 cts. per gallon, this company would be very glad to do such work as is mentioned at 3 cts. per square foot, or under the same conditions of work would lay a wire mesh and apply a gunite coating of ¾ in. at from 5 cts. to 5½ cts., and on either basis would expect these prices to pay a reasonable profit.

Such a great difference in figures could hardly arise from the difference between the costs of labor and material in Chicago and San Francisco.

I am writing you this letter in the hope that you will publish it and thus correct so far as possible the erroneous impression created by the article, and the fact that I desire to publish this letter should prove that gun work of this kind can be done at prices mentioned herein.

Yours very truly,
Chas. A. Cooper.
Pres., General Cement Gun Co.
Chicago, Ill., March 13, 1914.

(Answer to this letter has been made by Mr. Sessions as follows—Editors:)

To the Editors: Referring to the above quotation from my article in Engineering & Contracting of March 11, 1914: At the time the article was written the price given was that quoted by a local company for work that the writer had in view. The price was considered out of reach and the idea of using the cement gun was abandoned. It seems that the cement gun is now established under a new management here on the coast and that there is not so much of the idea to sell guns as to do a contracting business with them. The following prices have been quoted for work with this cement gun and as a matter of information and in justice to the cement gun people I would suggest that you publish them as a correction or addition to the article in question.

For a job of about 20,000 sq. ft. and materials, etc., at the following prices: Cement, per barrel $3.20; sand, $1.50 per yard on the job; water, $1.50 per day; freight each way, 25 cts. per hundred; railroad fare, $3.00 one way; reinforcing of No. 7-A Triangle Mesh Fabric over a backing of building paper, and mixture of one part of cement to three parts of sand; 1½ ins. thick, 11 cts. per square foot; 1 in. thick, 10 cts. per square foot; ¾ in. thick, 8 cts. per square foot.

The mixture of 1:3 is at the gun, there being enough loss of sand caused by rebound to make the mixture in place about 1:2½ to 1:2¾. One gun will do this job in about three weeks.

For 1,000,000 ft. of ditch work a price of 6½ cts. has been quoted for a lining 1 in. thick

Yours truly,
C. R. Sessions.
San Francisco, Cal., March 31, 1914.

Large Bridge to Be Constructed in Germany.—It is reported that construction will soon be begun of a railway bridge connecting Rugen, an island in the Baltic Sea almost due north of Berlin, with the mainland of Germany. This bridge, when completed, will be one of the longest in the world, exceeding even that over the Hwang ho, which is 10,740 ft. long. The structure will probably include a track for pedestrians, although none for road traffic, and when completed it will substantially shorten the journey between Berlin and Hamburg on the one hand, and Stockholm and Christiania on the other. The cost is estimated at about $5,000,000.

Engineering and Contracting

Devoted to the Economics of Civil Engineering Design
and to Methods and Cost of Construction

Volume XLI. CHICAGO, ILL., APRIL 29, 1914. Number 17.

Flood Prevention and Protection as Advocated by the Board of Army Engineers.

From time to time we have published articles treating of flood prevention and protection measures, and have commented, editorially, on this subject in several recent issues. In view of its importance to engineers it is instructive to note the conclusions reached by the board appointed by General Kingman, chief of engineers, U. S. Army, at the direction of Secretary of War Garrison. This board of army engineers, which went to the scene of the floods in the Ohio and Wabash valleys last spring to investigate and devise practical means of controlling flood waters in American streams, has just made its final report to Congress.

The report advises that the work be placed entirely in the hands of the War Department, to avoid the duplication of organizations and the expense of forming new ones. It also advises that the War Department be given control of non-navigable tributaries of navigable rivers. But it rejects as impracticable the Pinchot method of forestation of watersheds, a scheme which has been a bone of contention between engineers for many years. It is stated that no one method can be devised to fit the needs of all streams, but that each stream must be studied before any plan of flood control is adopted for it. Immediate work on the surveying of streams is advised, the authority to be included in the pending rivers and harbors bill. Among the rivers selected for immediate survey are the following: Wabash River and its tributaries; the Cheat River of Pennsylvania and West Virginia; the Tygarts, West Fork and Kanawha rivers of West Virginia; the Muskingum River and its tributaries and the Scioto River, both of Ohio; and the Maumee River of Ohio and Indiana. The estimated cost of these surveys is $80,000, and it is recommended that this sum be taken from the $250,000 item in the pending bill for the survey of streams. Among the streams included for future surveys are the Mohawk, the Delaware, the Susquehanna and the Brazos rivers.

In a letter accompanying the report Secretary of War Garrison summarizes the results of the board's investigations, the recommendations of which were adopted for his own report. In this he was joined by Gen. Dan C. Kingman, chief of engineers. Mr. Garrison cited in detail the various methods proposed for flood control, and made the following comments:

> (1) Forestation and special methods of plowing are not practical enough to make it profitable to consider their employment.
>
> (2) Reservoirs, storage and impeding, may be used to advantage, especially in smaller streams where sites may be acquired at a reasonable cost.
>
> (3) Moving structures of bulk out of a flood area gives prompt and certain relief, not only saving the property removed, but reducing flood heights and velocities by removing the obstructions. This method should be adopted so far as practicable and new obstructions should be prevented.
>
> (4) Auxiliary channels, parallel and cut off, may be used to advantage in certain cases.
>
> (5) Levees, when properly built, have been almost uniformly successful. They offer quick relief and are suitable for both local and extended uses.
>
> (6) No one method of flood control can be recommended to the exclusion of others. Local conditions vary so greatly that what is suitable for one stream is likely to be unsuitable for another.

Legislation is recommended (a) to amend the present law making it unlawful to obstruct the tributaries of navigable streams, this prohibition now applying only to navigable rivers, (b) to authorize the survey of the several rivers mentioned, and (c) to authorize the survey of other streams from time to time, as conditions may warrant.

The members of this board of army engineers are: Lieut. Cols. Francis R. Shunk and Henry Jervey, and Majors C. S. Bromwill, John C. Oakes, Frederick W. Altstaetter and Lewis H. Rand.

Emergency Financing of Water Supply and Sewerage Projects.

Many small cities abandon water supply and sewerage projects when the city officials learn that the estimated cost of the projected works exceeds the sum which can legally be borrowed to defray the cost of the contemplated improvement. Frequently the constitutional limitations governing a city's right to borrow for public improvement construction are permitted to deprive the city of utilities which the majority of the citizens want, and for which they are ready to pay. That a way is open to cities in this predicament is well illustrated by the methods employed in financing the additional water supply for Centralia, Ill., and the new sanitary sewerage system for Vincennes, Ind.

The facts in the Centralia case, which are fully set forth in an article published in the water works section of this issue, are briefly as follows: The municipal supply of water was obtained from a contaminated creek which went dry every summer. The city was unable to borrow enough money to pay for new works on account of the bonding limit fixed by law. It was deemed unwise to grant a water works franchise for fear the rates charged by the water company would not be paid by a railroad whose shops form the city's chief industrial dependency. A committee of citizens then organized a local stock company to construct new works and made an agreement with the city council to lease and operate these new works and to purchase them at their full cost. The stock company was capitalized at $150,000. Water department revenues were pledged to pay 6 per cent interest on the total annual indebtedness remaining from year to year. The city issued $50,000 in bonds in part payment of the cost of the works and agreed to issue $10,000 worth of new bonds each year and agreed, further, to raise a similar amount each year in general taxes to be applied to the balance of the debt. The methods employed by the committee in securing the capital stock subscriptions from local people and in disposing of the irregularly issued bonds are very interesting and are fully explained in the article to which we have referred.

The methods employed in financing the Vincennes sewerage system were fully described in this journal for March 11, 1914, but are here briefly reviewed. In Indiana three methods of financing sewer construction are ordinarily employed. They are: By appropriation from tax budget, by assessment of property benefited, and by bond issue. These methods were either impracticable or inadvisable under the local conditions, and were abandoned. The plan adopted was as follows: The council granted to an association formed for that purpose a franchise to build, operate, maintain and extend a well defined system of sewers. For the use of this sewerage system the city pays an annual rental. With the franchise as a basis bonds were issued by the sewer association. These bonds draw interest at 6 per cent and the annual rental is such that it will pay the interest and retire all the bonds at the end of 25 years. The city has the right to purchase the system, after 30 days' notice in writing, at prices which decrease from year to year as agreed at the outset. At a cost of slightly over $2 per capita annually for 25 years the city will secure sewer service and will then own the system. During this period the system will be properly maintained. This method of financing at Vincennes has been and promises to remain very satisfactory.

The methods of financial procedure here indicated may be applicable to conditions in other cities where improvements are desired, but where sufficient funds are not available through regular channels. Certainly these methods should be carefully considered before projected public improvements are abandoned on account of a city's constitutional limitations.

The Use of Manufacturers' Catalogues by Engineering Students.

It is a common practice of teachers in engineering colleges to advise their students to write for catalogues of machinery and supplies. This is done under the supposition that the students will study these publications and so gain knowledge of the character, uses, sources and prices of machines and supplies used in engineering construction. This kind of knowledge is always valuable and many times it is essential. We doubt, however, that under the present practice of students in collecting and using such catalogues the ends sought are attained.

The following statements relative to the present use of catalogues by students are based on a nine years' observance of the conditions discussed. It is a common practice among students to write for a considerable number of catalogues at one time. The requested copies are all received by the student within a few days. Coming, as they do, in considerable number, they present a rather formidable appearance to the student who is busy with his studies or with his other activities collectively known as "college life." A very natural result of these conditions is that the various catalogues are given but scant attention. Of course, there are some students who carefully and repeatedly examine publications of this character, but the average student feels that the procuring of catalogues is in itself so highly commendable and is such strong evidence of his alertness that their real uses are overshadowed by the considerations named and they are allowed to go practically unread.

It is rare indeed that a student keeps, until he becomes a buyer, the catalogues he collected during his undergraduate days. On graduating the student commonly disposes of, or at least stores, his textbooks in order to lighten the impedimenta which he must transport about the country with him in his pursuit of a livelihood. If such action is taken with books he has purchased one can readily see how unlikely it is that he will preserve manufacturers' literature which he has obtained without cost. In the light of the foregoing we conclude that neither the student nor the manufacturer profits from the present average practice of students in the use of manufacturers' catalogues.

Our plan for bettering the conditions here discussed is, briefly, that each department of the college should keep a file of catalogues of all machinery and supplies used in connection with the field operations for which the instruction given in the department is preparatory, and that students should be required to make certain prescribed uses of these catalogues as a part of their course of study.

To illustrate our suggestion let us suppose that a class of civil engineering students is taking up the study of water supply. The students, if our suggestion were in force, would consult the catalogues pertaining to this branch of engineering from time to time as directed by the teacher in charge. They would be directed to look up certain definite matters and to report their findings in a notebook kept for that purpose. The scope of the notes would, of course, be indicated by the teacher in accordance with the importance of the portion of the subject under study. Such a notebook, if kept in connection with all the subjects taught in the department would be of real value to the student in the early years following graduation.

The keeping of notes of this character should be encouraged as it constitutes a beginning of the preparation of the "handbook" which every engineer should compile for himself. This habit of keeping notes for future use can well be formed by the undergratuate under the guidance of his teachers.

Reconnoissance Surveys As a Feature of Extensive Highway Improvements.

There appears to be in vogue but two ways of securing preliminary information in regard to road construction in many sections of the country. One is to make an extensive survey, at a high cost per mile, of roads which may not be improved for several years or possibly not at all, and the other is to depend upon the judgment of the engineer.

The first method is excellent, but, unfortunately, cannot be followed very extensively, except in sections where ample funds are available and there is some possibility that the survey may have a future value.

The second method is in more general use. While it is quite probable that the engineer's judgment as to the best route to follow and the approximate cost of a proposed road improvement, even when based upon a merely superficial inspection which often is all the investigation he has an opportunity to make, is much superior to the ideas and opinions of others, yet there is no doubt that a more careful investigation, such as is afforded by a reconnoisance survey, would not only be a material aid to his judgment, but would in a large measure do away with the necessity of exercising it.

We believe that a reconnoissance survey, such as outlined in the article in this issue on Michigan practice in that respect, is an invaluable aid in planning extensive highway improvements. Accurate data obtained in this way furnish a means of comparison and a basis for estimating costs which are free from the element of chance which necessarily enters into any estimate which is not based on such data.

In this connection, the advisability of making such surveys to obtain information to be used as a basis for issuing bonds for highway improvements cannot be too strongly emphasized. Too often bonds are issued for road purposes with the idea of accomplishing work which later is found to cost much more than was originally intended. This usually leads to unjust criticism of officials in charge of work. The cause for this criticism would not have existed had sufficient preliminary data been available.

The method of procedure described in the article, to which we have made reference, has much to commend it. The field work is done rapidly and inexpensively, and the information obtained is full and of a useful nature. Surveys of this nature are valuable and we commend their use to road officials and highway engineers.

Roads and Streets

Methods of Making Reconnaissance Surveys for Trunk Line Highways in Michigan.

A reconnaissance or preliminary survey is made to obtain data which may be used to determine the most desirable route to follow for a proposed road and its approximate cost of improvement. Methods of conducting these surveys have been highly developed for railroad work but in the highway field they have not received the same attention. This is doubtless due in a large measure to the fact that in the construction of short sections of road such surveys are of small value. In the last few years, however, ther has come a demand for the improvement of through roads, and in selecting an economic route for such a road it is necessary that accurate information of utility and cost be secured. Moreover, with such data at hand the engineer and road commissioner is able to plan his work with much greater ease and more definite assurance of its ultimate cost. Surveys of this nature should also be made prior to issuing bonds for extensive road improvement. This is especially true where a large mileage of improvement is contemplated.

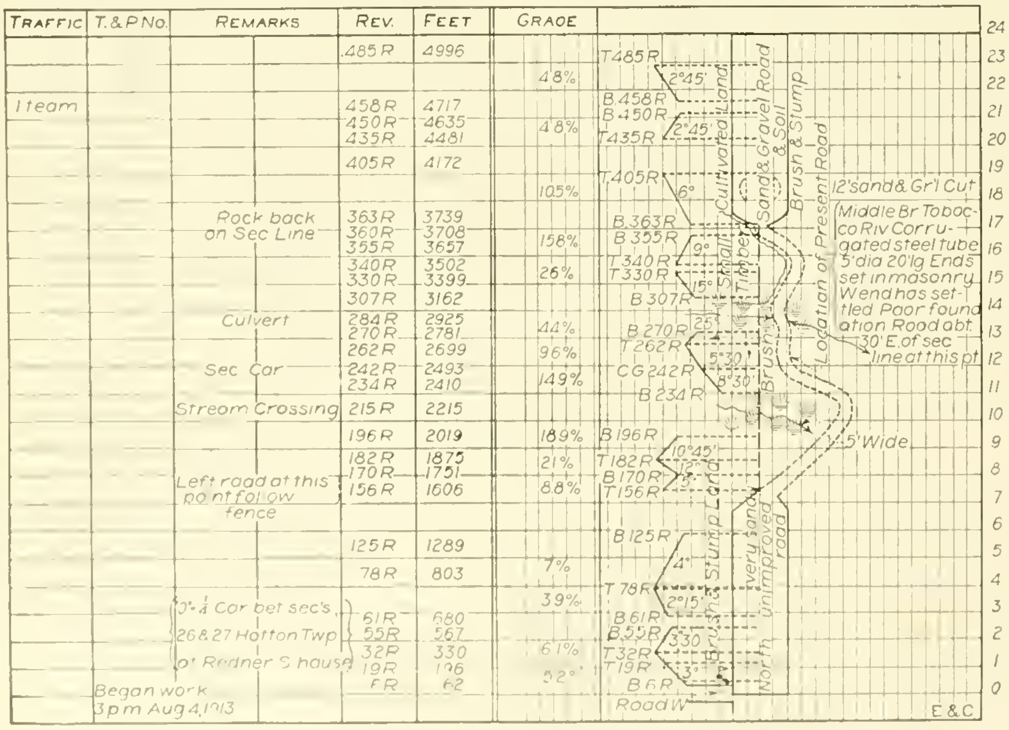


Fig. 1. Sample Page from Field Book of Reconnaissance Survey.

Mr. L. H. Townsend of the Michigan highway department in a paper before the Michigan Engineering Society has described the methods employed in that state for this class of work and we are indebted to him for the information contained therein

Division ten, section two, of the Michigan Trunk Line Highway Act reads, in part, as follows:

> Provided that on petition of two or more contiguous counties through their boards of county road commissioners, in counties under the county road system, or through their boards of supervisors in other counties, or upon his own action, the State Highway Commissioner shall make preliminary surveys and such other investigations as he shall deem necessary of one or more routes for State Reward Trunk Line Highways through such counties. After investigation the State Highway Commissioner shall submit to such county road commissioners, boards of supervisors, good roads district commissioners or township boards, as the case may be, maps showing feasible routes and approximate estimates of cost of building over each of said routes, when the said boards shall concur upon one of such routes which shall then be known as a part of the State Reward Trunk Line Highway System.

In a general way these routes were established, such as the one beginning on Michigan Avenue in Detroit and running via Wayne, Ypsilanti, Ann Arbor, Chelsea, Jackson, etc. But to know which road would be the most suitable to follow, it was necessary to have a reconnaissance made of one or more roads connecting the towns and then select the most feasible, as stipulated in the Act.

Preliminary Information.—In all cases where it was possible the highway officials of the county, township or good roads district were requested to submit to us the road or roads, in their respective communities, which they wished to have the trunk line highway follow. Where this was done, and there was but one choice and no reconnaissance was applied for, we generally adopted that road as a portion of the trunk line system. But where they failed to give a definite choice or there were several contending routes, or where the route chosen by them did not seem feasible to us, a reconnaissance was made.

At the beginning of my services with the state highway department the method of handling the reconnaissance work had not been decided upon. Owing to the fact that I was connected with the topographic branch of the United States Geological Survey for several years, I was very much in favor of the use of the plane table. I still contend that the topography as sketched by that survey gives a clearer idea of the country traversed than any other way, but since a novice is liable to misinterpret topography and would be misled in translating any maps which we would wish passed upon we decided to resort to some other method.

The use of an auto was considered but on account of the speed with which it covered the ground and the inaccuracy of the measurements taken, any information gathered would be too general for even a reconnaissance.

Our final decision was to use a wheel or pace traverse and keep a running set of notes of all information gathered.

Equipment and Field Notes.—A good box compass, clinometer, tape and camera were the instruments used; the buggy wheel or pace was employed as the means of determining distances. It was rather a case of being a close observer than in doing accurate instrument work. One had to be on the lookout for such things as the character of country traversed, whether it was timber, cut over land, clearing or improved farms; approximate location of buildings, giving kind; condition of road grade, and if improved in any degree—the width, alignment and drainage; kind of soil on roadway, whether sand, clay, loam or swamp (notation being given of the distance from the starting point to the several changes); location, condition and size of ditches, creeks and streams that might serve as outlets for water from the roadway; measurements of culverts and bridges with condition and location, photographing all truss bridges and others that might need to be repaired or rebuilt; bearings of roads traversed with compass, also compass bearings of intersecting railroads, large streams and roads unless at right angles; noting intersection with corporate limits of cities and villages, if possible. The most important features to be noted were the length and grade of the hills gone over on the road in question. The traffic was also noted and divided into three classes, single rigs, double rigs and automobiles. Although this did not even give a fair estimate as to the amount of travel on the road, it nevertheless showed that it was traveled to a greater or less extent and unquestionably had an influence on the decision of the route to be followed. Consideration was also taken of the available road material such as gravel, stone, etc.

A sample page from a field note-book is shown in Figure 1.

Mapping.—In mapping the reconnaissance work we considered it the wiser plan to have these drawings conform as nearly as we could to the working plans for the finished road. By doing this they showed the more essential features, such as profile, alignment and drainage of the road, and were more easily handled by the officials to whom they were submitted.

A fair sample of this portion of the work is shown in Figure 2—a portion of the reconnaissance made in Clare county. The small sketch shows the routes followed. In this county there were several different roads which could be taken and used as a portion of the trunk line highway. The heavy line shows the one in preference. These maps were drawn to such a scale (1,000 ft. to an inch) that ten miles of road could be conveniently placed on a sheet 20x36 ins. in size. The width of the road was exaggerated in order that such things as cuts, fills, ditches, etc., occurring within the limits of the road could be shown clearly. All information gathered was placed on, or adjacent to, the plan of the road except the profile which was shown below.

Extent and Cost.—Nearly 1,200 miles of road were covered by a reconnaissance during the past summer, several hundred miles of which have already been mapped and the remainder is being compiled as the work demands it. The cost was approximately $1.50 per mile for the major portion of the field work.

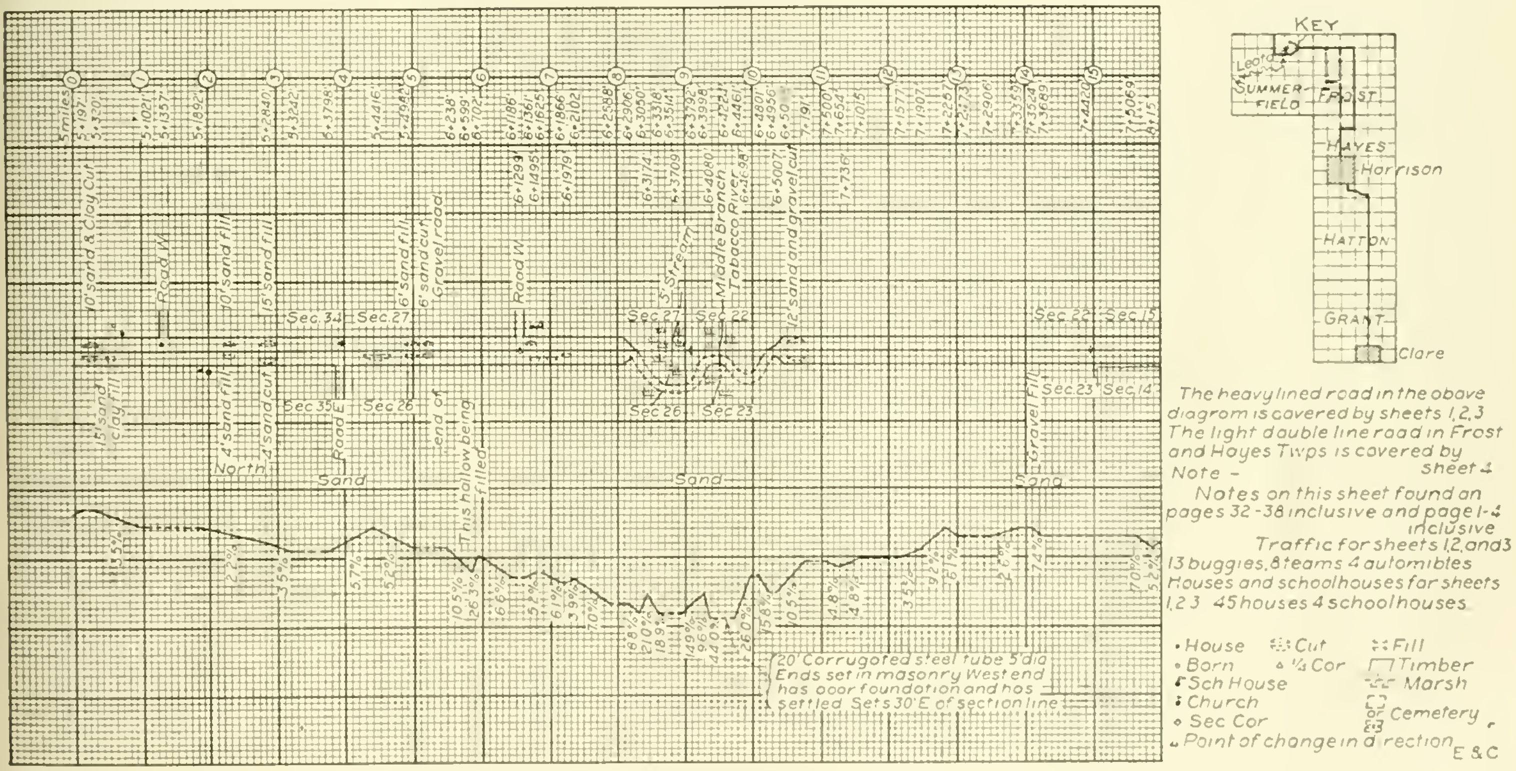

KEY
Leota
SUMMERFIELD
FROST
HAYES
Harrison
HATTON
GRANT
Clare
Sec. 34
Sec. 27
Sec. 35
Sec. 26
Sec. 22
Sec. 23
Sec. 15
Sec. 14
Road W
Sand
North
This hollow being filled
Gravel road
Stream
Middle Branch Tobacco River
12' sand and gravel cut
Gravel fill
20' Corrugated steel tube 5' dia. Ends set in masonry West end has poor foundation and has settled. Sets 30° E of section line
The heavy lined road in the above diagram is covered by sheets 1, 2, 3
The light double line road in Frost and Hayes Twps is covered by sheet 4
Note – Notes on this sheet found on pages 32-38 inclusive and page 1-4 inclusive
Traffic for sheets 1, 2 and 3
13 buggies, 8 teams 4 automibles
Houses and schoolhouses for sheets 1, 2, 3 45 houses 4 schoolhouses
House
Born
Sch House
Church
Sec Cor
Point of change in direction
Cut
¼ Cor
Fill
Timber
Marsh
Cemetery
E & C

Fig. 2. Method of Mapping Reconnaissance Survey in Michigan.

Destroying Teredos by Electricity.—A new method of destroying teredos, a pest which has caused enormous damage in the destruction of piles used in wharfage construction in north Pacific coast ports, particularly in Vancouver, Seattle and Tacoma, by turning an electric current into the piles, is being tried in Seattle. These pests have so far successfully defied all attempts to eradicate them from the timbers supporting the various structures along the water front in the Puget Sound and British Columbia ports, and a means of destroying them will prove a great saving to individuals and companies maintaining docks and wharfage property on the coast.

Summary of Street Traffic Conditions in a Number of Large Cities.

The rapid development of American cities in the past two decades has brought the problem of providing for present and future traffic squarely before the engineer. A consideration of the problems of other cities is often helpful in the solution of immediate difficulties and the summary below, taken from the report of the Manchester (England) Tramways Committee, as presented by *The Surveyor* of London, is of interest in that it gives the viewpoint of men who are dealing with foreign traffic conditions which are often much more complicated than those in this country.

An investigation was made in part, for the purpose of determining so far as possible (1) the probable increase in traffic in the next 20 years; (2) provisions necessary to diminish the increased congestion in the central streets and facilitating traffic by means of (a) widening of existing streets and the construction of new streets; (b) the use of an arterial center for street car traffic, and (c) the construction of subways.

GENERAL PROBLEM

As cities have grown, due to the gradual increase in population, and as the riding habits of the people have developed, especially in the last few decades, it has become more and more difficult to make the channels of passenger transport equal to the ever growing demands. In every city the main factors affecting the problem are much the same. There is a constant migration of the population from the inner to the outer zones, and this is greatly accelerated by every extension and improvement in the means of transit. Suburban districts rapidly become urban, and new suburban districts in the outlying areas are always in progress of formation. Smaller towns and districts on the outskirts are being drawn into closer touch with the cities, and a greater community of interests is being established over a gradually widening area.

Every extension of the means of transit into the outer zones adds to the difficulties of handling the traffic in the central zone, and in time the congestion becomes so acute that the city authorities realize they are face to face with the inevitable traffic problem. This condition of things arises sooner or later in all large cities, but some cities have been confronted with the problem at an earlier date than others owing to the narrowness and bad lay-out of the streets in the older parts of the city, to the absence of suitable arterial roads, and to the presence in the central zones of a large amount of ordinary vehicular traffic, which, if it is slow-moving, tends to retard the general flow of the traffic.

NEW YORK.

Some interesting features presented by the transit facilities now in operation, and those which are under construction in New York, are noted in the report. Greater New York, with a population of about 5,000,000, and an area of 324 square miles, embraces Manhattan Island and the boroughs of Brooklyn, Queens, Richmond and Bronx. Manhattan Island, the original city prior to the inclusion of the adjoining boroughs within the city area, is the most densely populated piece of territory in the world, and the crowding of huge buildings for business purposes into its limited area has created a transportation problem of the most complex kind.

The most interesting feature of the existing transportation facilities is the underground railway in Manhattan. This subway is owned by the city authorities, and is leased to the Interborough Company. It is constructed under the streets of the city, and as near the surface as possible. The usual depth of the station platforms from the street levels is about 25 ft., and the approaches are by short flights of steps. The nearness of this subway to the street surface, and its easy accessibility as compared with the deep-level "tubes in London, with their lifts and long subterranean passages, at once illustrates the advantages from the passengers' point of view of shallow underground railways as against deep-level "tubes." The subway has four tracks two for express services and two for local services. It was originally built to carry 400,000 passengers per day, but it is now carrying over 1,000,000 per day, and the traffic is still increasing.

The problem as to the best method of meeting the present demands and making provision for the future occupied the attention of the public service commissioners for several years. Ultimately they prepared a scheme which, as regards breadth of conception and thoroughness in all its details, is unsurpassed. The entire scheme, when completed, will add 333 miles of rapid-transit lines to the existing 296 miles, making a total of 629 track miles. The total cost of the new scheme when completed will be about $340,000,000—a figure about equal to the amount invested in the existing rapid-transit lines in New York. Thus the total investment on these undertakings, as outlined, will reach $685,000,000.

PHILADELPHIA.

The most striking traffic feature in this city is the small mileage of rapid transit lines as compared with what exists in other large cities in the United States. The rapid transit lines run between the business district on the east side of the city and the suburban district on the west, and they cater only for the flow of traffic in these directions. The lines in the business area are of the shallow subway type. In the outer area they are elevated. A special feature of the underground facilities is that, at the sides of the tracks laid for the rapid transit trains, tracks are laid for street cars, which enter the subway at certain convenient places and handle the local traffic The system is designed so that the tramways shall act as tributaries to the rapid transit lines.

BOSTON

The chief feature which characterizes the methods employed in Boston to meet the passenger transportation requirements is the excellent way the various means of transit have been devised to supplement one another. The authorities have endeavored to eliminate waste of facilities by not creating the unnecessary competition which accompanies the paralleling of different means of transit. The car lines, both on the surface and underground, and the rapid transit lines are all operated by one company—the Boston Elevated Railway Company. This method of having only one operating authority for all the means of transit in a city has much to commend it. Under proper conditions and safeguards it leads to the best results.

CHICAGO.

In this, the second largest city in the United States, the population is widely spread, and every endeavor has been made to insure that the transit facilities should keep pace with the public demands. Developments are now being proposed in the direction of further rapid transit lines to relieve the congestion of the surface lines which, in certain streets in the heart of the city, is becoming very acute.

PITTSBURGH.

The question of the provision of rapid transit in this city will have to remain in abeyance for some years until the street car lines have been modernized and brought nearer to their maximum efficiency for dealing with the passenger transportation requirements of the city. Ultimately, rapid transit lines will have to be provided.

NEWARK, TORONTO AND MONTREAL.

In each of these cities many points connected with the construction and management of the street car systems were interesting and instructive, but there was nothing of importance from which lessons might be drawn in relation to the special investigation. In each of the cities there are troubles with a certain amount of traffic congestion, and suggestions have been made in the Canadian cities that relief might be provided by the introduction of rapid transit facilities, but the subject has not yet taken any practical shape.

PARIS.

The city of Paris is the only place where practically the whole of the surface transit facilities— street car lines and motor omnibuses—are operated by one authority on a large scale, and it was thought, therefore, that an unbiased opinion might be obtained as to the merits and demerits of the two systems from the point of view of the management. On this question being put to the officials there was a strong pronouncement in favor of the street car lines, mainly on the ground of less cost in operation.

BERLIN.

In addition to rapid transit lines, there are the State railways within the city of Berlin. One line runs through the city, and the other is a circular line—the ringbahn.

The large network of street car lines in Greater Berlin is operated by a company—the Grosse Berliner Strassenbahn. There are a number of important streets in the heart of the city from which tramways are excluded, and surface transit in these streets is provided for by motor omnibuses.

VIENNA.

Vienna is the only large city on the Continent where the street car system is owned and operated by the municipality, and this, added to the fact that the city has just had a thorough investigation made with the object of introducing a number of new rapid transit lines to meet the city's growing transportation needs, makes the Vienna problem very interesting.

At the present time there is a rapid transit line—the Stadtbahn or State Railway, partly elevated and partly underground—which was originally constructed for State purposes, and is therefore not well laid out for dealing with local passenger transportation. It is a steam-driven line, but is just about to be electrified. It forms a connection between a number of main line railway stations.

The proposed new rapid transit lines closely follow the line of the flow of traffic, and it is intended to construct them on the shallow subway plan. With a view of obtaining the fullest information on the subject of rapid transit lines, exhaustive inquiries have been made on behalf of the city into the methods of construction and operation of the American rapid transit lines.

The motor bus services have not, so far, made much headway. The municipality, however, is gradually introducing them, and considerable development may be expected. Experiments are now being conducted with a view to the adoption of covered tops for the upper decks of the buses.

A proposal has been made to the city that shallow subways should be constructed in order that the street cars now excluded from the central area might be brought underground within that area, and thus provide the through services which are much needed. This proposal, however, has not found favor with the city authorities, they being of opinion that if subways are to be constructed they must be operated as rapid transit lines by high speed trains, otherwise the cost of their construction is prohibitive.

HAMBURG.

The city of Hamburg has a large network of street car lines, and, in common with other cities, the traffic congestion along certain arterial lines became so acute that in recent years steps had to be taken to meet the difficulties. The street car company suggested the construction of subways, but after a full investigation the city authorities negatived this proposal, and determined to construct lines for high speed trains—rapid transit lines; these are partly in shallow subways and partly elevated. The lines form a circular route, but the flow of traffic along the northern portion of the route is comparatively small, and scarcely justifies its existence.

The city provided the necessary capital for the construction of the lines, and the operating company provided the capital for their equipment. The company has to pay to the city a certain percentage of the gross receipts, but otherwise is not responsible for meeting the fixed charges on the construction costs.

LONDON.

The defect of London's rapid transit system is that it has not been laid out in accordance with a comprehensive plan so as to meet the requirements of London as a whole. It has been laid out piecemeal in accordance with the plans of different private companies, and there has been no permanent authority which has exercised control over the planning and development of the system. This method of handling one of the most difficult problems a city is faced with can only end in failure to secure the most satisfactory results.

Individual companies and authorities promoting and operating the various means of transit, without the controlling and regulating influence of a central authority acting in the public interests, is a condition of things which everyone who has studied the question should not have permitted to grow up, because it produces anything but the best results.

From the passengers' point of view, the main advantages of the motor bus over the street car in London are:

(a) That its mobility enables it to run practically anywhere. Narrow and congested streets are not shut against it, and it gives the through service, which the street car is unable to do.

(b) It can maintain a slightly higher average speed than the street car, and this appeals to the increasing desire on the part of the public for more rapid transit.

(c) It can set down and pick up passengers at the crossings, thus avoiding the danger which is experienced in boarding and alighting from street cars.

There is no other city in any part of the world so favorably circumstanced for the working of motor buses as London. It has the best paved streets in the world. It has an immense population, which is growing at an estimated rate of 100,000 per annum. The riding habits of the people are increasing rapidly. They are still a long way below those of New York, so there is ample scope for increased traffic—new traffic—and not traffic necessarily drawn from other channels. And, as has already been pointed out, motor buses supply what the tramways are not permitted to do—they give through services to all parts of the metropolis. The rapid transit facilities of London are not all that could be desired from the passengers' point of view. This, again, is favorable to motor-bus operation. The motor-bus competes fairly well with the rapid transit of the "tubes." The time occupied from point to point at the street level is not much less by "tube" than by motor-bus, except for long distance traffic.

CONCLUSIONS.

(1) That in all cities and towns there is an immense potentiality in the riding habits of the people. Everywhere there is a continual growth in the number of journeys per head of population, and the provision of new or improved transit facilities accelerates this growth in a very marked degree.

(2) That in planning city transit facilities a wide outlook is all important; any schemes carried out on narrow or confined lines ultimately result in serious losses to the community from every point of view.

(3) It is essential that every city should have a permanent authority directing the initiation and carrying out of all schemes for new transit facilities, in order that there may be continuity of policy, and that future needs of the city and the surrounding districts over a wide area are properly looked after. It is also desirable that all the means of passenger transportation—both surface and rapid transit —should be centralized under one management. If the management is not that of the city itself, then it should be subject to control and regulation by a permanent authority appointed by the city.

(4) That, in planning underground lines for rapid transit, the aim should be to supplement the surface transit facilities; that the rapid transit lines should be laid so as to follow as closely as possible the direction of the flow of the greatest volumes of traffic, and the surface lines should be adapted so as to act as feeders to the rapid transit lines at all convenient points.

(5) That, on account of the great initial cost, the construction of underground rapid transit lines cannot be justified unless there is a very large volume of traffic to be dealt with; that, speaking generally, underground subways for street cars are an unjustifiable expense owing to the comparatively small number of passengers it is possible to pass through them by single or double unit cars. Underground lines, if they are to be placed upon a paying basis, must be worked by the high speed trains operated at a very close headway. This demands a great density of traffic.

(6) The motor buses have proved themselves to be an exceedingly valuable means of surface transit, especially under conditions such as exist in London and Paris; but the question of their general adaptability for conditions which exist in other cities, and the part they will play in the future, either in supplanting or supplementing the existing street car facilities, calls for a careful analysis of their advantages and disadvantages from the local point of view.

Some Reasons for Failure of Certain Types of Bituminous Surfaces.

Contributed by Allen Hoar, Civil Engineer, Long Beach, Calif.

That there have been many failures of bituminous macadam and gravel roads and bituminous pavements, is not to be wondered at. These types of roads and pavements have been adopted generally throughout the country and subjected to all kinds of traffic conditions. They have been used both wisely and unwisely, without regard to kinds and requirements of local traffic conditions, and have been built of a variety of materials and by as many different methods. In fact bituminous macadam and gravel roads are today undergoing the rigorous tests that waterbound macadam underwent a few years ago, and if given half a chance, bituminous pavements will be found to meet the present needs of traffic conditions just as efficiently as did the waterbound macadam ten years ago.

There are many different types of bituminous macadam and bituminous gravel pavements, all, however, being some slight variation of the penetration, the hot-mix, or the petrolithic methods, and each having its staunch supporters.

General Causes of Failure.—The causes of failure of bituminous pavements are numerous, and may be considered as arising from three principal causes: (1) materials, (2) method of construction, and (3) condition and character of the original surface or foundation.

The failure of bituminous pavements due to the condition and character of the original surface, is often the result of improper care, and failure on the part of those in charge, to see that the surface is properly prepared to receive the application of the binder. Many cases are noted in the resurfacing of old pavements, where no provision has been made for the proper care of pot-holes and ruts, or where the surface has been left in a dirty condition, due to either the accumulation of dust or to loose dirt resulting from the character of the original surface. Before attempting to resurface an old road, all loose dirt and dust should be thoroughly swept off with wire or reed brooms, and all ruts and pot-holes should be cleaned out and patched. Often it may be necessary to scarify the whole surface to a depth of one or two inches and bring it to cross section before applying the bituminous material.

Failure is often due to the selection and use of materials, the physical properties of which are not at all suited for the purposes and needs of the work in hand. Liquid asphalts containing large percentages of volatile constituents, which require a long time to set, have been used for top dressing and seal coats. This results in keeping the pavement in a soft and sticky state for a long period of time. Coal tars and paraffin base petroleums, rich in free carbons, have been used as binders, dependent for success upon adhesive qualities which they lack. Bituminous materials, chemically and physically unstable in character, have sometimes been used.

Just how much of a chemist the engineer must make of himself, may be a question to some, but the engineer must know his materials, and if he would devote more time to a consideration of the chemical and physical properties of the materials which he uses, there would be less failures.

Failures are also due to improper application of the binder, to the use of either too large or too small amounts, and to lack of a sufficient covering of rock screenings or coarse sand. Probably the greatest percentage of failures has come from the use of too much bituminous material, resulting in a thick, spongy mat, easily cut and picked up by horseshoes and iron tires, and which creeps and becomes rutted and wavy under traffic. During the hot days of summer, this fault causes a disagreeable condition due to bleeding.

Poor Application.—Construction failures are due for the most part to wrong methods of applying the binder, resulting in an uneven distribution of the material, and thereby preventing a uniform mixture of the stone and bitumen. The heavy spots remain soft for a long period of time before setting, and allow a shifting of the stones under traffic. This causes fine cracks, which soon develop into ravelling and finally disintegration. The thin spots soon show up by ravelling, due to an insufficient film of the binder to cover and adhere to the stones. To obviate these effects, the binder must be sprayed on hot under pressure. The binder is sometimes not hot enough to allow it to penetrate far into the aggregate. Much of the success of liquid asphalt is due to the stability of its physical properties, and any asphalt, the properties of which are changed to an appreciable extent by heating, is unstable, and therefore little better than a coal tar or paraffin base petroleum. The heavier grades of liquid asphalt should be heated to almost the flash point. The lighter grades do not require so much heat. Coal tars and paraffin petroleums are to a more or less degree unstable, and above certain comparatively low temperatures, the formation of carbenes takes place, which, when continued to any appreciable degree tend to make the bitumen to some extent brittle and to kill its binding qualities.

Wrong Size Stone.—Many unsatisfactory pavements result from the use of wrong sizes of rock, either too large or of too uniform size. These features prevent both a mechanical locking of the aggregate, and a uniform penetration of the binder, taking place. Poor assorting of sizes often permits segregation during spreading, and results in an ununiform pavement of varying stability. The use of a wet aggregate will usually result in preventing a complete adhesion of the binder to the stone, with consequent unsatisfactory results.

Foundation.—Many failures in this type of pavement are due to faulty foundations. Too much attention can not be given this important part of the construction.

Methods and Costs of Snow Removal in Scranton, Pa., in 1914.

The amount of snowfall during the past winter furnished a good opportunity to secure data on the cost of snow removal. Scranton, Pa., with a population of 129,867 in 1910, provides an example of the average conditions which exist in small cities. The information given here is abstracted from a paper by J. G. Hayes, director of public works for that city, presented before the Philadelphia Snow Removal Conference.

Removal by Hand.—During the months of February and March last, the city of Scranton, as well as other cities in the east, suffered probably from the most severe snowstorms in its history. In some instances the snow was drifted from eight to ten feet in depth at street intersections, and with the regular force of employes, numbering 250 men, intersections were opened where the traffic was most important, piling the snow on the nearest corner, and later in the day removing such accumulation by means of city wagons and dumping it into the nearest sewer manhole. If, however, a manhole was located at the street intersection which we were cleaning, the snow was removed by shoveling it direct into this opening, thus saving the second handling. This, however, could not be done in all cases on account of the fact that in some instances the diameter of the sewer below this manhole would not be of sufficient size to carry the accumulation away. The work of removing snow in this manner, while eventually successful, is very slow and expensive. This statement is proven by the fact that to remove the snowfall in February, 1914, it cost the city of Scranton approximately $3,500 for opening street intersections alone, and to render the principal streets of the city somewhat passable. A conservative estimate of the cost of removing snow in this manner would be at least 85 cts. per cubic yard.

Removal by Flushing.—The topography of this city, however, is an asset towards snow removal by flushing it into the sewers. This arrangement was never experimented with by former administrations of this city, but it was later found to be successful.

In order to remove this accumulation by the flushing method the most important features to consider are the sewerage systems and the water pressure. A constant water pressure of 75 lbs. per square inch exists at each fire hydrant throughout the city, and sewers are of ample area to convey this accumulation when flushed into it, to the river, which is the outlet of our sewerage system. The Scranton Gas and Water Co. receives, on a contract basis, $12,000 per annum for the water consumed through all fire hydrants in the city, thereby rendering the removal of snow by the flushing method inexpensive from a standpoint of water consumed. The method of removing snow in this manner was as an experiment, and tried first on Lackawanna avenue, the principal business street of the city. Fire hose was attached to a nearby hydrant and the course of the water was directed to the open catch basin, from which the grating had been taken and the trap removed, allowing a 10 in. opening direct into the sewer. The men employed on this work were stationed from ten to fifteen feet apart and, with the aid of necessary implements loosened and removed the snow and ice which had accumulated. In 13 hrs., with 24 men, 14,440 sq. yds. of pavement was cleaned. Notwithstanding the fact that the snow in places was as high as 8 ft., but averaging the height of snow at 3 ft., which is very conservative, it would leave 43,320 cu. yds. of snow and ice removed at a cost of less than 25 cts. per cubic yard. In addition to the small cost it is readily seen that such an arrangement eliminates the regular spring cleaning, which, under the old system, required the ice to be dug away from the pavement and carted off in wagons.

On Penn avenue, another one of the principal thoroughfares of this city, the same force, working eight hours, removed 22,420 cu. yds. of snow at a cost of a trifle over 21 cts. per cubic yard.

Rivers and Harbors

Method and Cost of Constructing Sand Core Levee in the Sacramento Valley.

Contributed by R. G. Clifford, District Engineer, Natomas Consolidated of California.

The extensive reclamation work in the Sacramento Valley in California being done by the Natomas Consolidated of California was outlined in ENGINEERING & CONTRACTING of Jan. 21, 1914, under "Current News" and a description of the sand-core levees mentioned will undoubtedly prove interesting to those engaged in reclamation work.

There are about 81 miles of new levee being built, enclosing 87,000 acres of rich land which extends along the Sacramento, Feather and Bear rivers and is divided into two main districts by a cross canal just below the mouth of the Feather River. The sand-core levee begins at this cross canal levee and extends down along the east bank of the Sacramento River 17.7 miles to the American River just above the city of Sacramento.

Levee failures along the Sacramento in the past have chiefly been from the wave wash occurring during the occasional extreme flood stages of short duration. Other causes are over-saturation during extended high water of levees built with clayey materials and excessive percolation through underlying strata or between the made-ground and the original ground surface, causing a sliding or upheaval of the levee.

Economical construction of these extensive levees necessitates the use of the materials immediately at hand and these consist of a deep sand in the river bottom and stratified sedimentary deposits of clayey silt along the banks. The average height of this 17.7 miles of river levee is 15 ft. and approximately 4,500,000 cu. yds. of material were necessary to construct a bank of sufficient stability to be absolutely safe under conditions of maximum flood.

Most of the recent levee work has been done with large clam-shell dredges having 5 or 6 cu. yd. buckets, which pile the river sand up along the banks after clearing the timber and brush from the site. These sand banks are very slow in accumulating any growth and are thus left unprotected from scour and wave wash unless brush or tule mats are provided at considerable expense. A heavy growth of cotton wood, willow and black oak lines the river throughout its length and it was to take advantage of this natural protection to current and wave action that first suggested the use of a levee built with a suction dredge.

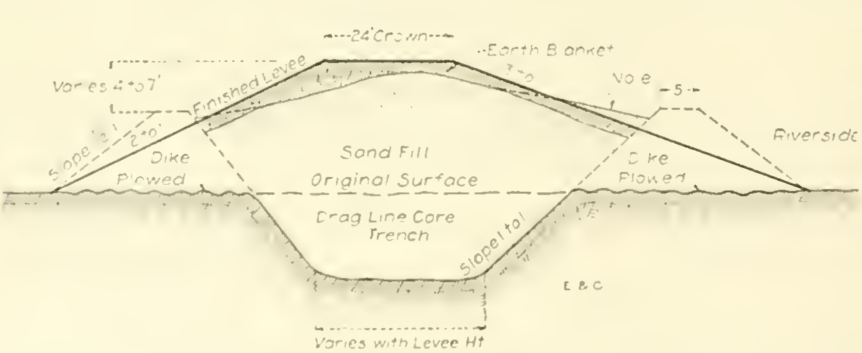


Fig. 1. Cross Section of Sand Core Levee Showing Sequence of Construction Operations.

The levee was located with its center line 150 ft back from the natural river bank except where the distance was less than 800 ft to the existing levees across the river when this minimum flood plane width was used.

DRAG LINE EXCAVATORS.

Retaining dikes, Fig. 1, are thrown up first by drag line excavators mounted on trucks having 16 wheels in all running on two standard gage parallel tracks. The booms are 100 ft. in length and 3½ and 4½ cu. yds. Bucyrus and Page buckets are employed in the varying material encountered. These drag lines backed down the cleared right of way, digging a deep cut-off trench of sufficient size to furnish material for the two retaining dikes on either toe of the finished levee. The crown of these dikes was kept about 6 ft. below the finished top of the levee and was made 5 ft. wide with a natural slope of about 1½ to 1. The total yardage included in this drag line work for the river levee is 1,224,400 cu. yds. The operation of Drag Line No. 2, which did 60 per cent of the work, is given in Table I.

The actual cost of excavating this 743,056 cu. yds. of core trench material and placing it in the two dikes is divided as follows:

Item—	Cost.	Unit.
Labor of operation	$24,013	$0.0323
Fuel	8,645	0.0116
Superintendence and engineering	2,800	0.0038
Moving and erection at beginning	2,440	0.0033
Repairs	11,900	0.0160
Total	$49,798	$0.0670
20 pct. annual depreciation on $28,717 worth of equipment	5,743	0.0077
Grand total	$55,541	$0.0747

The efficiency of operation increased very materially after the crews became trained to handle this particular character of construction. The levermen worked 6 hours on and 12 hours off, making an average of 8 hours per day.

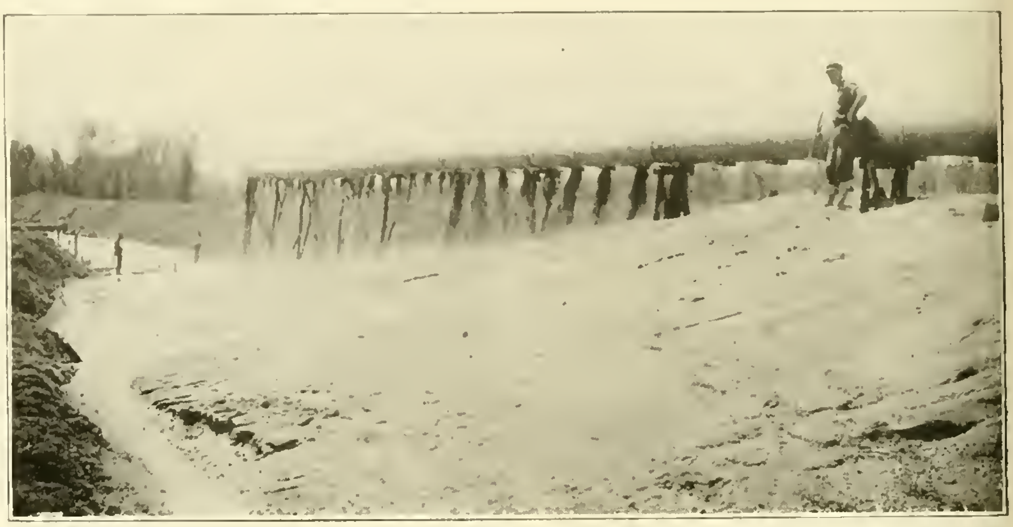


Fig. 2. Filling in Sand Core Using Pipe with Bottom Openings or Traps.

SUCTION DREDGE.

The suction dredge used was built by the Yuba Construction Co., of Marysville, Cal., under the direction of Mr. S. L. G. Knox, now general manager of the Natomas Consolidated of California. The suction pump is 20 ins., driven by a 650 hp. triple expansion steam engine, crude oil at 85 cts. per barrel (first cost) being used for fuel. The hull of the dredge is 105 ft. x 35 ft. and 9 ft. deep. The limits of digging are 7 ft. minimum and 36 ft. maximum, although the average has been 18 to 32 ft. The lift has been from 20 to 30 ft., with 1,400 ft. average length of pipe used. The crew consisted of 3 levermen, 3 engineers, 2 firemen, 2 deck hands, all on the dredge, beside a shore gang of 34 men total for the two 12-hour shifts. The levermen worked 6 hours on and 12 hours off, making an average of 8 hours per day.

TABLE I.—OPERATION OF DRAG LINE EXCAVATOR.

Date.	Operating hours.	Digging time. Hours.	Digging time. Pct. total.	Yardage moved.	Cu. yds. per hr. digging.	Fuel oil.	Cu. yds. per bbl. oil.	Cu. yds. per lin. ft.
1912—								
May	426	307	72	32,260	105	364	89	10
June	720	617	86	54,930	89	732	75	10
July	726	578	80	65,600	97	797	82	11
Aug.	741	593	80	49,560	84	767	65	11
Sept.	720	560	78	54,820	98	746	73	12
Oct.	744	597	80	85,220	113	932	91	10
Nov.	720	604	84	104,772	174	1,067	98	11
Dec.	702	575	82	111,092	193	1,080	103	11
1913—								
Jan.	744	627	84	120,128	192	1,103	109	13
Feb.	66	51	78	7,349	156	75	106	13
Sept.	311	232	74	34,137	147	740	46	18
Oct.	258	166	64	22,588	136	242	93	25
Total	6,884	5,507	80	743,056	135	8,645	87	13

The dredge worked downstream so as not

TABLE II.—OPERATION OF HYDRAULIC DREDGE.

Date.	Operating hours.	Pumping time.		Yardage moved.			Fuel oil.	
		Hours.	Per cent total.	Total.	Cu. yds. per hour.	Cu. yds. per lin. ft.	Total.	Cu. yds per bbl.
1912—								
September	218	104	48	28,430	273	28	300	95
October	744	414	56	97,480	235	36	881	110
November	720	333	46	78,130	235	41	799	98
December	744	404	54	65,613	163	31	964	68
1913—								
January	744	189	25	24,270	129	40	501	49
February	672	334	50	63,406	190	35	765	83
March	744	421	57	99,556	236	30	909	109
April	720	474	66	171,522	362	35	1,445	119
May	744	504	68	248,280	493	32	1,709	145
June	720	442	61	200,230	454	31	1,562	128
July	744	422	57	121,280	288	29	1,536	79
August	744	550	74	190,358	346	33	2,000	95
September	720	514	71	222,345	433	36	1,995	111
October	744	573	77	176,119	308	49	2,108	84
November	720	438	61	133,575	305	43	1,857	72
December	288	246	85	132,915	541	49	1,102	121
Total	10,730	6,262	59	2,053,509	328	35	20,436	100

to require outside motive power for moving. The pipe was extended across the intervening space between the river and the levee center and so-called pocket pipe was supported on rough bents for an average distance of 700 ft. on a slight down grade, as shown in the accompanying illustration, Fig. 2. Temporary wooden baffles 12 ins. high kept the stream at the discharge end toward the center of the levee to prevent washing of the side dikes. The pockets are merely openings in the bottom of the pipe, opened or closed as desired by means of simple sliding gates operated by the attendant. The sand drains itself readily and would build up on slopes of from 1 on 10 to 1 on 4, depending on the fineness of the sand and the amount of silt present. The operation of the suction dredge for the 2,053,509 cu. yds. handled to date is given in Table II.

The cost of placing this 2,053,509 cu. yds. of sand core is divided as follows:

	Cost.	Unit.
Labor of operation	$ 53,054	$0.0258
Fuel	20,436	.0100
Superintendence and engineering	7,117	.0034
Repairs	27,727	.0135
Total	$108,334	$0.0528
20 per cent yearly depreciation on $105,300, cost of outfit	28,100	.0137
Grand total	$136,434	$0.0665

Since there were 13 cu. yds. per lineal feet of drag line work against 35 cu. yds. of suction dredge work, the average cost of each cubic yard of levee untrimmed would be $0.07, including the assumed 20 per cent depreciation charge on equipment. This cost included the plowing of furrows parallel to the core trench underneath the side dikes to further prevent percolation between the original ground surface and the levee.

To compare with a levee built by a clam shell dredge, the side slopes being the same but with no core trench excavated, the cost of the 13 cu. yds. of dragline work would have to be added to the cost of the suction dredge yardage and the unit price would be $0.091 instead of $0.07

The justification for this additional expenditure is found in the efficacy of the earth blanket on the levee for furnishing a ready foothold for shrub and grass growth and the advantage of the core trench in breaking up the line of percolation and doing away with danger of the levee sliding on its base. It is also evident that to take advantage of the protection afforded by the natural growth along the river it was necessary to use the hydraulic fill type of levee, which in turn necessitated the use of the dikes. The presence of the sand in the core is the only certain way of preventing dangerous burrowing by gophers and other small animals.

In addition to the above levee cost there is a charge for pulling up the sediment in the dikes so as to cover the levee faces, as shown in the cross section illustrated. This covering sods readily and is soon comparatively water tight, while the growth starting on it without delay aids greatly against wave wash. Little of this trimming has yet been done, but so far has cost about $3,000 per mile, or about $0.012 per cubic yard of material in the levee, the work being done by teams. Since a road 24 ft. wide was to be built on the top of this levee, most of this trimming cost would apply to any form of levee constructed.

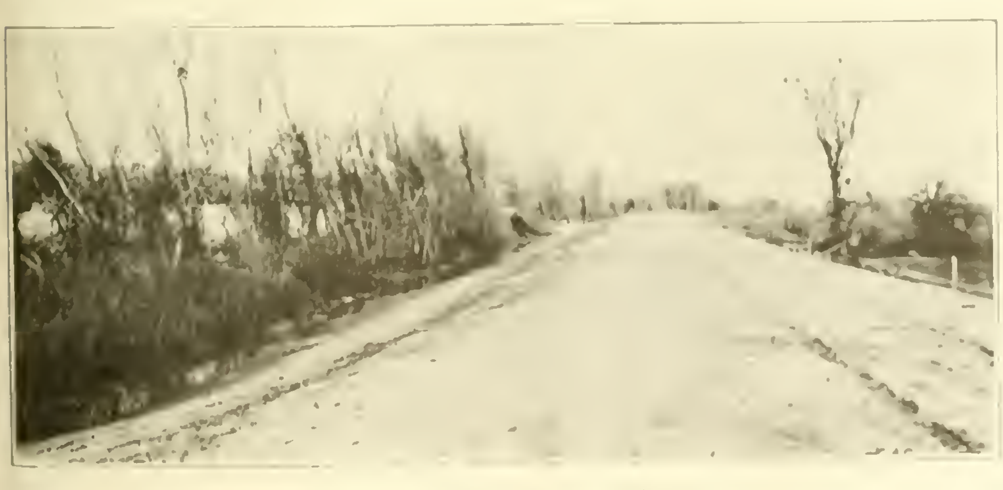


Fig. 3. Completed Sand Core Levee.

Comparing this type of levee with the usual dredger levee built, the cost is probably a trifle higher since contracts can be obtained for similar work for $0.07 and the cost plus 20 per cent depreciation is probably a little lower. As to stability, cost to maintain and ultimate safety to the thousands of acres enclosed, there is every reason to suppose that the sand core levees constructed are far superior to any other type so far built along the Sacramento River. One of the illustrations, Fig. 3, gives the condition of the finished section after the high water of 1914 had subsided and the protecting effect of the natural thick growth of willows and oak is evident

The rapid reclamation of all lands along this river will confine a vast volume of water that heretofore has overflowed into the extensive swamp land basins in the valley and even with the proposed weirs and by-passes, all the levees will receive severe tests in the next few years, so that it is likely to be a case of the survival of the fittest, and it is believed that these levees described will absolutely guarantee the land owners in this district against all floods.

Mr. Emory is chief engineer for the Natomas Consolidated Company and is responsible for the designing and successful construction of this type of levee.

Some Accounts of Dipper Dredge Performance on the New York Barge Canal.

Contributed by Emile Low, C. E., Buffalo, N. Y.

The construction work on the improvement of the New York State Canals, popularly known as the Barge Canal, is now so far advanced that the Hon. John A. A. Bensel, State Engineer, fixes the date of the final completion of the work as May 15, 1915, the usual opening date of the canals for navigation. The first actual work of construction was performed on April 24, 1905, at Fort Miller, in opening a temporary canal on the Champlain section. The first work on the main channel was done at Waterford, Erie Canal, June 7, 1905. Should the prediction of the State Engineer be realized, ten years will have been occupied in the building of this great waterway.

In no other public work, except the Panama Canal, have quantities reached such high figures. The required excavation will amount to 110,000,000 cu. yds., of which over 84,000,000 cu. yds. had been removed prior to July 1, 1913. In addition to the excavation, nearly 9,000,000 cu. yds. of embankment have been built. Roughly speaking, the excavation amounts to about one-third of the cost of the Barge Canal, or about $31,000,000.

The Barge Canal contracts provide but one price for all excavation, a lump sum for each cubic yard of material excavated of every name and nature, no attempt being made to classify the different materials excavated, as earth, rock, hard pan and what not, the notable exception being Contract No. 12, where the excavation is divided into four classes of material.

Much of the excavation was done by hydraulic dredges, supplemented by dipper and ladder dredges. It is the purpose of this article to give some information relative to the performance of the dipper dredges, which, though not so extensive as that of the hydraulic dredges, still comprises a large amount of excavation on scattered sections of the canal.

CONTRACT NO. 1.

The first work of this character was done in the Hudson River, on Contract No. 1, Champlain Canal, and extending from Northumberland to Fort Miller, and from Crocker's Reef to about one-half mile south of Fort Edward, a total length of 7.075 miles, the contract being held by the Empire Engineering Corporation and awarded April 18, 1905. The original estimated quantity of excavation was 913,500 cu. yds., and the contract price 57¾ cts. per cubic yard, without classification.

The present Champlain Canal parallels the Hudson River between Waterford and Fort Edward. At the latter place the Hudson bends to the west. The lower southern portion of the canal is built along the west bank of the river, crossing the same at Northumberland, and then to Fort Edward occupying the east bank The Barge Canal lies in the bed of the Hudson River, except at two localities—near Schuylerville and between Fort Miller and Crocker's Reef—at which points short land lines have been built. In other words, the Barge Canal is a canalization of the Hudson River, the river being made navigable by building a dam about three miles north of Waterford, using four existing dams built by private enterprise for power purposes, and erecting another dam at Crocker's Reef. To provide the required depth of 12 ft. of water, the various pools had to be deepened by dredging, and Contract No. 1 is a typical example of this class of work

Two dipper dredges, known as the "Peconic" and "Pontiac," were built for this work, and the hulls were brought to the site of operations by way of the Champlain Canal; the machinery being shipped in by rail and then installed. In order to pass the present locks of the state canals, the hulls were built to these lock dimensions, namely: 18 ft. beam, 97 ft. long and 6 ft. draft. To provide stability when working, suitable pontoons were attached, one on each side. The dippers have 3⅓ cu. yds. capacity.

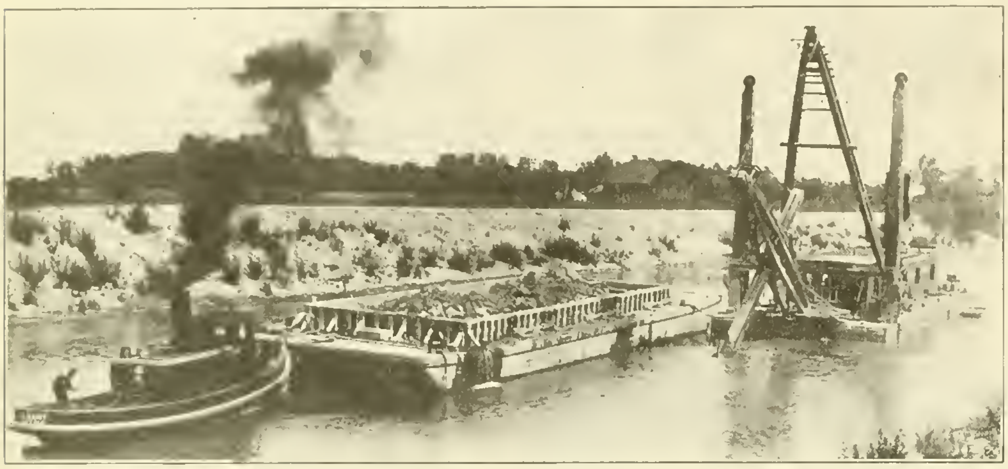


Fig. 1. Dipper Dredge "Buffalo" on Section No. 19, New York State Barge Canal.

Generally the excavation was of an earthy and silty nature, but between Crocker's Reet and the north end of the contract there were three rock reefs. To break this rock, a Lobnitz rock breaker was installed. In addition to the plant named, the contractor had in commission two tugs and six scows, besides five locomotives and two traveling cranes. Dredging operations were begun in July and August, 1906. In the beginning of the work, the dredged material, after having been placed in dump scows, was towed by tugs to an unloading dock, clammed out by Hulett buckets attached to traveling cranes and deposited into dump cars and transported to spoil banks by locomotives. Later most of the material was towed to dumping grounds along the shore and deposited. Occasionally a small amount was taken to the unloading dock and disposed as stated above.

The following tabulation shows the yearly output of the two dredges:

Year ending—	Cu. yds.
September 30, 1906	47,261
September 30, 1907	384,810
September 30, 1908	215,225
September 30, 1909	25,924
September 30, 1910	11,172
September 30, 1911	4,814
Total	689,206

During the fiscal year ending September 30, 1907, the two dredges worked 24 hours daily, continuously, except during the winter months of December, January, February and March. The working conditions for the year 1908 were about the same. It will be noted that the quantity dredged for the year 1909 was small. This was due to the fact that a large percentage of the excavation was broken rock. During this period the dredge "Pontiac" only worked during September and October, 1908, when it was partly dismantled and shipped to the Western Division, on Contract No. 64, via the Champlain and Erie canals. During the next two years only the dredge "Peconic" was in commission, working generally one 8-hour shift per day. The dredged material was practically all Hudson River shale, broken by the Lobnitz rock breaker. The excavation was completed September 19, 1911, at which time the dredge "Peconic" was dismantled and with other plant shipped to Buffalo, N. Y.

The breaking of the rock, which occurs mostly in vertical layers, was commenced on July 20, 1906, on the northerly reef of the contract. To September 30, 1907, 25,180 cu. yds. of rock were broken by the rock breaker. During the following years the machine worked intermittently, being much of the time out of commission, mostly on account of breakage of the points of the ram. There are no data at hand of the workings of this machine, but it is estimated that during the season of 1909 about 10,000 cu. yds. were broken. As there was no classification of material, one excavation price covering it all, there was no incentive to keep the quantities of earth and rock separate. Of course, the approximate quantities of each were known to the estimating engineer at the time of fixing the composite price, as is required by the Barge Canal law.

The output for the year 1907 was comparatively good, it averaging nearly 1,000 cu. yds. per day of 24 hours for each dredge, representing an income of about $560 per day, which, however, is very much smaller for the rest of the time while work was in progress.

CONTRACT NO. 26.

Contract No. 26 is for dredging a channel in the Hudson River, at Fort Edward, and adjoining the north end of Contract No. 1, previously described. The engineer's estimate was 172,000 cu. yds., the contract being held by the Lake Erie Dredging Co., of Buffalo, N. Y., and the price for excavation being 34¾ cents per cubic yard, the material being mainly silt, with no hard material. The dredging plant arrived at the site of the work on June 10, 1908, having been brought from Buffalo via the Erie and Champlain Canals. It consisted of a 1¾-cu. yd. dipper dredge, two dump scows and one small canal tug. The dredge, which was an old one, was like those of the Empire Engineering Corporation, of Erie Canal lock size, viz., 18x97x6 ft., with pontoons for stability while dredging.

The dredge began operations on July 13, 1908, dredging a channel to and in the spoil area or dumping grounds, which was separated from the river proper by building a round timber bulkhead. To pump the dredged material from the channel into the spoil area, a pumping plant, consisting of a 14-inch centrifugal pump, an 80-HP. engine, with boiler of approximately 100 HP., was assembled and installed on a small scow, 20x40 ft. in size. This plant for a portion of the time did not handle the material as well as expected, due to the large amount of sawmill refuse mixed with the sand and gravel. Operations were continued to December 10, 1908, when the plant was laid up for the winter, and operations resumed April 27, 1909. The same method of disposing of the spoil was followed as during the previous year, but it had soon to be abandoned on account of the sawmill refuse clogging the pump. The pump was superseded by a traveling derrick operating a Hayward clam, the construction of which was begun on May 7, and operation on July 10, 1909.

Due to a change in contract No. 27, adjoining on the north, the quantity of excavation in contract No. 26 was reduced to 115,200 cu. yds. Operations were continued to December 11 1909, when the plant was laid up for the winter. Work was resumed on April 25, 1909, and continued to June 10, 1910, when the contract was completed.

The dredged material in dump scows, after having been spoiled in front of the crib bulkhead, and placed in the spoil area by a clam shell, was leveled to the top of the crib with drag scrapers. The yearly operations were as follows:

Year—	Cu. yds.
1908	27,707
1909	65,664
1910	8,552
Total	101,923

CONTRACT NO. 19.

Contract No. 19 is for prism excavation in Tonawanda Creek, between the mouth of Ellicott Creek at the "Tonawandas" and the Sulphur Springs guard lock near Pendleton, the length of the contract being 12.46 miles. This contract is held by the Great Lakes Construction Company of Buffalo, N. Y., and was awarded November 26, 1906. The engineer's estimate for excavation is, for rock, 240,000 cu. yds., and for all excavation 2,842,000 cu. yds., the contract prices being 60 and 17½ cts., respectively. It might be inferred that the material is "classified," but this is not the case, as the rock excavation occurs at the eastern end of the contract, and the *all* excavation, generally clay, sand and gravel at the western end, the line of demarkation between them being well defined. Various excavation plants were employed on this contract, a Lidgerwood cableway, a Williams revolving derrick, clam shell dredges, a hydraulic dredge and a dipper dredge. Only the operations of the latter will be touched upon here.

This dredge called the "Buffalo" is 86 ft. long, 29 ft. 6 ins. beam, 8 ft. sides; the hull is constructed of the best grade of long leaf yellow pine. The main engine is 12½x15 ins. double cylinder, Bucyrus make. The swinging engine is 10x10 in. double cylinder. The boiler is the Scotch marine type, 8 ft. 4 ins. in diameter, 10 ft. long, with a working pressure of 150 lbs. The pinning up engines are 7½x7 ins. double cylinder, Bucyrus make. The bow anchors are of steel, 24 ins. square. The stem anchor is made of oak, 18 ins. square. The dredge is equipped with a 2½ cu. yd. hard digging dipper and a 3¼ cu. yd. soft digging dipper. The dredge will make 23 ft. of water. The cost was about $35,000. The present Erie Canal on contract No. 19 is a canalization of Tonawanda Creek, secured by building a dam across the same, at its mouth just before entering the Niagara River. This dam preserves the level of Lake Erie, the water surface of Tonawanda Creek being originally 6 to 7 ft. lower than the lake, depending on water fluctuations caused by winds. As the Barge Canal joins the Niagara River at the "Tonawandas" the original level of Tonawanda Creek will be restored, by taking out the dam mentioned above. As the prescribed depth of water in the Barge Canal is 12 ft., this, coupled with the present depth of the Erie Canal, 7 ft. and some allowance for over-depth and low water, dredging to a depth of 20 ft. and more was required in excavating the Barge Canal prism.

The work of the dipper dredge at the start was mainly excavating from the prism, the material being deposited in dump scows, which were towed to points near the shore at the spoil banks and dumped, to be redredged by two clam shell dredges, having booms of 80 to 100 ft., respectively, and the dumped material then rehandled into spoil banks.

Later on the dipper dredge was rebuilt and was used to dredge the hard material, encountered at the bottom of the prism, which at times verged close to hard pan. This hard material was dumped in various places along Tonawanda Creek, and later rehandled by the hydraulic dredge "Niagara." The following tabulation shows the output of the dredge for a number of years:

Years—	Cu. yds.
1908	132,687
1909	28,100
1910	81,873
1911	30,500
1912	168,192
Total	441,352

The following tabulation shows the cost of labor for running the dredge for a double shift of 16 hours:

	Per mo.	
1 captain	$157.50	$157.50
1 runner	137.50	137.50

cranesmen	106.50	213.00
oilers	77.50	155.00
firemen	77.50	155.00
deckhands	67.50	270.00
scowmen	67.50	270.00
watchman	60.00	60.00
blacksmith	75.00	75.00
		$1,493.00

As the average output of the dredge was nly about 10,000 cu. yds. at the contract rice of 17½ cts., amounting to $1,750, it is pparent that this barely covered the expense f wages and coal.

Concrete Protected Steel Sheet Pile Bulkhead Wall.

(Staff Article.)

A bulkhead wall of protected steel sheet iles forms a part of the work included in e planned barge canal terminals at Erie Basin, Buffalo, N. Y. The new wall is some 320 ft. long and connects new pier and bulkhead walls of concrete capped timber cribs with an old masonry wall of an adjacent slip. A diagram section of the sheet pile wall is given by Fig. 1 (A). As indicated the wall is located instream and the land in front of it is to be dredged out to provide 13 ft. depth of water.

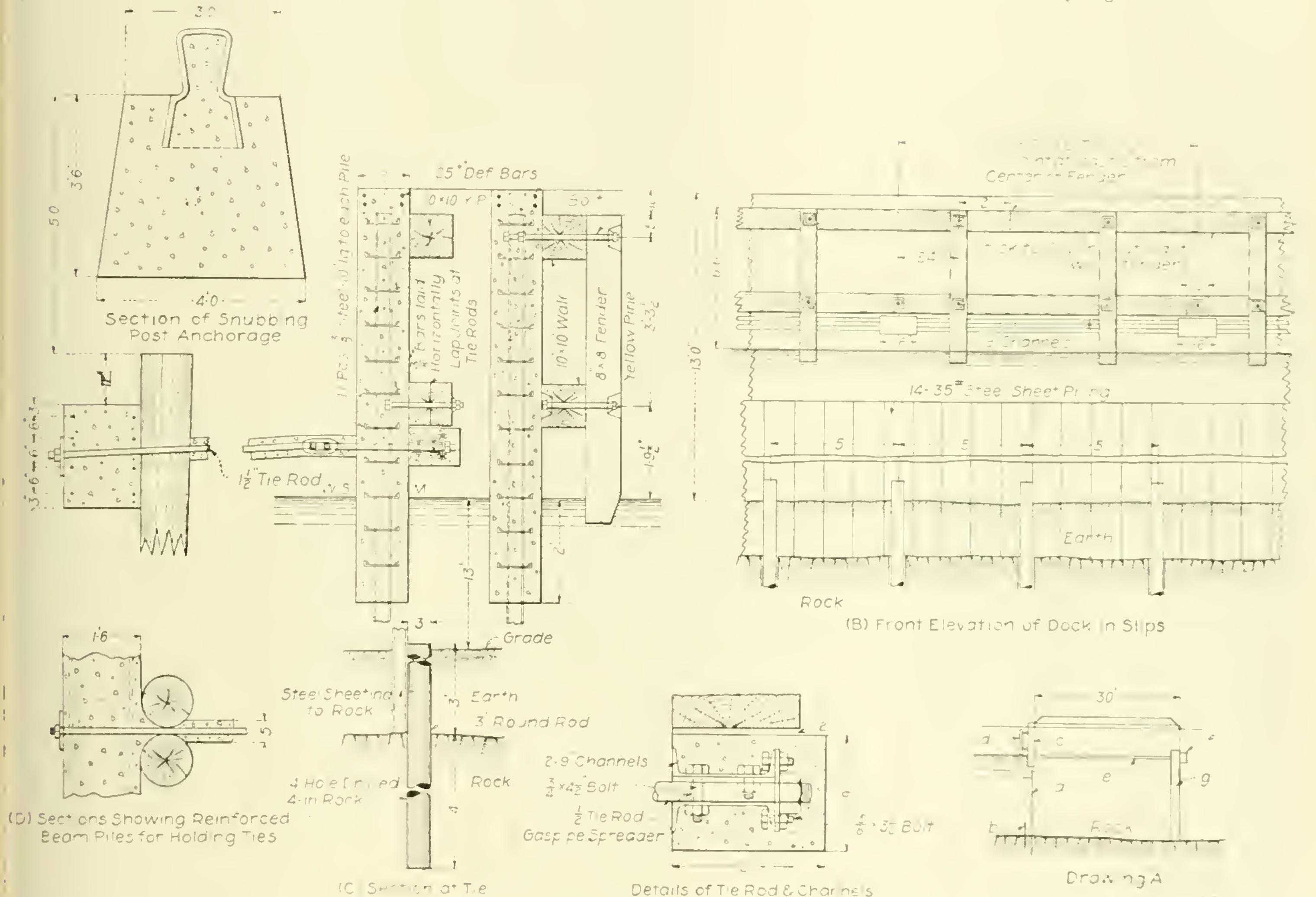


Fig. 1. Details of Concrete Protected Steel Sheet Pile Bulkhead Wall, Barge Canal Terminal, Erie Basin, Buffalo, N. Y.

The details of the structure as described are shown by Fig. 1, in which (B) is a front elevation of the wall, (C) gives sections at and between fenders and (D) shows the anchor beam and pile arrangement. All the essential structural details are indicated by the drawings and a brief summary of the material and construction requirements follow:

The steel sheet piling is to be of medium steel and special care to secure exact alignment is to be exercised in driving it. The holes for the transverse reinforcing bars, fenders, anchor butts and tie rods will be drilled after the sheeting is driven. The concrete embedding the top of the sheeting and the tie rods and that for the anchor beam will be a 1:2:4 stone mixture. All reinforcement will be open hearth steel deformed bars.

Referring to Fig. 1 (A), *a* indicates the line of steel sheeting driven to rock bottom and held by the anchor bars *b*; the top of this sheeting is incased in a concrete slab *c* provided on its water face with fenders *d*. The wall as described is anchored back into the shore by tie rods *e* connecting with a horizontal reinforced concrete girder *f*, held by timber piles *g*. The ties *e* are spaced 11 ft. 8 ins. apart and the piles *g* are driven in pairs on each side of a tie rod where it is fastened to the beam *f*.

The wall as described was designed in the office of the state engineer, John A. Bensel, Albany, N. Y.

Water Works

Method of Financing the Additional Water Supply for Centralia, Ill.

Impelled by necessity the citizens of Cen- alia, Ill., found a method of financing the ditional water supply for the city, although nstitutional limitations for a time made it pear that funds for the new supply could t be secured. The method by which this as accomplished is of interest as showing hat a community can do in emergency financing of public utilities. Our information is rom a paper read before the Illinois Water upply Association by Mr. C. D. Tufts, president of the Centralia Water Supply Co.

Some 20 years ago the city took over a pumping plant of the Illinois Central Railroad located on Crooked Creek, two miles north of the city, and established a city water works. The Illinois Central, being the largest consumer of water, secured a 20 year contract under terms which fixed the price paid at approximately 1¾ cts. per 1,000 gals. for water pumped two miles. The creek, for the past few years, went dry every summer, and the quantity of water drawn from it was always poor.

The municipality was unable to raise funds to secure an adequate supply, on account of the unusual cost, and the constitutional limitations. The council, as well as the people, were unwilling to turn to a private corporation, the chief objection being that the most important industrial dependence of Centralia is the Illinois Central Railroad, with its extensive shops and terminals, which we feared would not be willing to pay the water rates that would be charged by a private corporation. No local capitalists were to be found willing to take over the water business and guarantee low rates, and we could not expect outside capitalists to load themselves up with our unfortunate predicament.

Confronted with this imperative demand, the city council was totally lacking in both the concerted energy and the tools and materials necessary for the task before it. The mayor called a meeting of citizens in the summer of 1909 to unload on them the water problem. In that meeting a committee of seven was raised and given fully authority to devise and carry into execution plans to provide an adequate water supply. After several months of preliminary skirmishing the committee decided to follow a plan of financing a supply that had previously been adopted in Mattoon under somewhat similar conditions. Prof. Habermeyer of the University of Illinois, was employed by the city on the committee's recommendation, to make surveys and recommend a suitable site. He recommended a site comprising the valley of two streams emptying into Crooked Creek, some eight miles east and up-stream from the pumping station.

Following the recommendation of Prof. Habermeyer, the committee secured the employment of Mr. E. F. Harper of East St. Louis, to make preliminary surveys and estimates of capacity and cost. He reported that a dam 800 ft. long and 30 ft. high would impound a body of water 24 ft. deep at the dam and containing approximating 1,000,000,000 gals. The submerged area was estimated at 250 acres, and the watershed 8 square miles. A spillway 150 ft. in length was provided, and also about 8 miles of 20-in. wood conduit was laid. The estimate of cost, necessarily only approximate, was $125,000.

Up to this time it had been considered absolutely impossible to raise by public subscription in Centralia an amount in excess of $25,000. Yet the spirit of "We Must" had taken possession of the committee, and it settled down to work with determination.

A stock company was organized, capitalized at $150,000, and an agreement entered into with the city council on the terms of a contract under which the city obligated itself to lease and operate the reservoir and pipe line, and to purchase these at their exact cost. The revenues of the water department were pledged to pay 6 per cent interest on the amount expended, and the city also obligated itself to issue $50,000 in bonds in part payment of the purchase price, and to issue $10,000 of new bonds each year, so as to make an annual payment on the balance. The contract further provided that the city is to levy $10,000 in general taxes each year to pay off that much of the original issue of bonds, so that the original amount of $50,000 stands with $10,000 paid off and $10,000 issued each year. This roundabout method is necessary on account of the legal limitations as to issuance of bonds and taxation.

With this agreement between the city council and the company, the next thing was to get the money. The committee called in the presidents of the two banks and made out a list of "prospects," setting opposite each person's name an amount that in the combined judgment of the committee and the bankers he could afford to subscribe. The banks consented to take the notes of these persons without other security than the company could furnish. The plan was to ask them to subscribe certain sums to the capital stock of the company and to assure them that the company would obtain the money on their notes without calling on them to do more than sign. The shares of stock signed for were deposited with the banks as collateral security, and the banks furnished the money necessary for the construction.

The list of names started with $5,000, and from that on down to $1,000. No subscription was taken for less than $1,000. The lists were made up one Saturday afternoon, and the chairman of the committee announced that the capital stock must all be subscribed the following week.

On Monday afternoon the newspapers contained leading articles explaining and commending the project and carried at the head in good-sized type the names of those who had subscribed $5,000. We did not call on any one to subscribe the first three days, except those on the $5,000 list, and we refused to accept from any of them a subscription for less than $5,000. Each day we published the names of all who had subscribed $5,000, with a story of the progress of the campaign. Our method was so effective that before we had finished with the $5,000 list, several who were on for less sums voluntarily came forward and signed for $5,000. On Thursday night we had eighteen $5,000 subscriptions. By that time we had the ground broken for quick work on the men of lesser means, and when we quit work Friday night we had $133,500 subscribed, which at that time we hoped would be sufficient. There were fewer than 40 subscribers.

In order to understand the task that confronted the committee, it must be understood that never before had an individual subscription to a public enterprise been made in Centralia of more than $1,000, and never before had an attempt been made to raise more than $25,000. It was only the spirit of "We Must" that made it possible to raise $133,500 in four days.

Then the Illinois Central had to be reckoned with. The contract of the company which fixed the price of Crooked Creek water at 1¾ cts. per 1,000 gals. had three more years to run. It was impossible for the city to meet its new obligations to the Centralia Water Supply Co. unless the railroad company would make a new contract at a fair rate. When the representatives of the city and the water company called on Vice-President Park and informed him a rate of 6 cents would be demanded, he was somewhat startled, but remembering the previous drouths, he told us to go ahead.

The bond issue of $50,000 had to be submitted to a vote of the people, but that gave us little trouble. There was some talk of graft, but that did not bother us much. As a matter of fact, we undertook all the responsibility and the moral and financial hazard of financing and constructing a project which finally cost $158,000, and no officer or director of the company received a dollar of compensation either directly or indirectly. Even the lawyers in the company gave their professional services without charge.

The farmers who owned the land needed for the reservoir suddenly became much opposed to the project. They held indignation meetings at the country school houses, and threats were heard that when we went out to dicker for their land they would meet us at the township line with shotguns. The land was worth $40 an acre, but at the school house meetings a price of $150 was fixed, and nearly all the owners joined in an agreement to demand that much or die in the attempt. The company had the legal right to condemn, which the farmers knew, but fortunately for us, they didn't know that if we had been compelled to condemn we would be delayed so long we could not get the dam constructed in time to catch the next spring's rains. This would have meant another year of drouth for Centralia, which would have cost the city more than all their land was worth. We secured one tract for $65 an acre, then bought the land needed for the dam for a price slightly higher. We were then in a position to let the contract and start the work. If we happened to submerge some land before we had acquired it, there would be no damages greater than the value of the land. Every acre of land required, the pipe line right-of-way and every necessary settlement was secured without any litigation whatever. This illustrates another point: if your lawyers are working without compensation, it is rarely necessary to go to court. We paid $65 an acre whenever we took only what we needed and $50 an acre where we were compelled to take the entire farm.

The bond brokers would not take the bonds issued by the city in part payment on the contract, on the ground that no such transaction had ever run the gauntlet of the Supreme Court, and the lawyers would not hazard a guess as to what the court of last resort would do to our plan if it got hold of it. The company took the bonds and held them in its treasury as assets for security of its stockholders and the banks that had loaned the money on their notes. After the work was completed and accepted by the city, the local banks took the bonds and credited the $50,000 on the stockholders' loans. In addition there have been three payments of $10,000, thus reducing the indebtedness of the city to the company to $70,000. The total cost was $158,000, but there were several items of income which reduced the actual net cost to $150,000. The city now pays six per cent on $70,000, and this money is paid to the banks which hold the notes. Quite a large proportion of the stock is now held, however, by subsequent purchasers who have taken it as an investment. These were people whom we couldn't interest when we were trying to get the stock subscribed in the first instance.

The work of construction of the dam and the pipe line, and clearing of the reservoir site was begun in the early fall of 1910, and completed in time to catch the spring rains of 1911. The water has a natural fall of about 25 feet beside the depth in the basin, which carries it by gravity through 20-in. wood pipes to the pumping station, where it is delivered direct.

At times of heavy rains, there has been a flow of fully a foot of water over the spillway of the large reservoir, and Centralia is now certain to enter upon a dry season with a depth of 24 ft. at the dam. Up to this time the surface of the water has never gone below 18 ft. at the dam, notwithstanding the consumption of the Illinois Central has more than doubled since the company has rebuilt its terminals and shops in Centralia. This new work of the railroad could not have been undertaken at Centralia without a dependable supply of water was assured, and its importance may be realized when it is stated that the new work of the railroad at Centralia has meant an expenditure up to the present time of $2,000,000, and a very considerable increase in the working force.

The revenues of the city water department were more than doubled with no material increase in the cost of operation. When the original system was installed, the city authorities, without looking to the future, issued bonds and laid the service pipes without a special assessment. This policy has been followed since, and there have been very few extensions of the water mains. Now the water department has a profit of from $1,00 to $1,500 a month, and the water mains are being extended to meet the demands of outlying property owners.

Notes on the Selection, Application Operation and Limitations of Centrifugal Pumps.

The present article points out the general characteristics of centrifugal pumping machinery with reference to the best types of pump for a particular duty, enumerates the services for which these pumps are especially adapted, describes general operation methods and the limitations of such pumps i actual practice. The information given i taken from a paper before the Ottawa branc of the Canadian Society of Civil Engineer by J. W. W. Drysdale, Jr., of the firm o Drysdale & Co. of Glasgow, Scotland.

GENERAL CHARACTERISTICS OF CENTRIFUGA PUMPS.

Centrifugal pumps, including turbine pumps are regulated by the following general laws

1. The quantity pumped when running a constant speed varies with the total head Total head includes suction lift, delivery an friction heads. At constant speed with increase in head the discharge will be decreased Conversely, reduction of head causes an increase of discharge.

2. The efficiency of a pump running at constant speed varies with the output, and wil reach a maximum under a certain conditio of head and discharge. A pump which is running at this point is said to be giving norma output. For any given speed, there is definite maximum possible head which th pump can overcome, and the normal outpu is nearly always a little below this point.

These general laws have limitations. Fo

instance, the sides of the impeller are revolving in what is generally termed dead water, and hence if the speed of the impeller is increased, the disc friction is increased until the amount of power absorbed in this manner becomes too large a proportion of the total power absorbed by the pump. When this occurs it is necessary to increase the number of impellers in the pump. One impeller delivers into the next and so on, the head being obtained in a number of stages. In addition, care must be taken that cavitation does not occur. Cavitation may take place either on the suction side or on the delivery side of the impeller. The effect of cavitation is generally overload on the driving machine and unstable delivery by the pump, and in serious cases the pump will stop working altogether. On the suction side, cavitation is caused chiefly by the suction lift being carried beyond the practical limit. On the delivery side, it is most often met with in centrifugal pumps with guide-vanes, that is, that class of pumps generally known as turbine pumps. In these pumps it is generally due to badly designed guide-vanes, or to the pump being used to deliver a quantity of water far in excess of the normal output for which the pump was designed.

From the characteristic curve of a well-designed centrifugal pump, it may be seen that a good efficiency can be maintained over a wide range. Thus, for example, from such a curve it may be seen that the capacity can be varied from 70 per cent below normal to 120 per cent above normal with a fall of only 5 per cent in the normal efficiency. Under such circumstances a typical curve shows that the maximum possible power taken by the pump is only 17 per cent greater than normal. This being so, it follows that a reasonably well built motor will be capable, running at constant speed, of taking charge of any duty which the pump may be called on to perform at that speed. As a rule all that is necessary is to design the pump so that the maximum overload does not exceed about 10 per cent to 15 per cent or even 20 per cent, as any motor with a good rating will stand this overload without damage.

An important point to notice in connection with centrifugal pumps is that there is no possibility of damage to the pumps in event of the delivery valve being closed during pumping operations.

It is possible by suitable modifications in the internal design of pumps to alter their hydraulic features to suit particular requirements. Thus pumps can be built in which the variation of the quantity with head is large, and others with small variation, but pumps with large variation of quantity are susceptible to a heavier overload than those with small variation. It is not possible to combine maximum quantity variation with minimum overload, but while minimum overload can only be combined with moderate quantity variation, a very fair compromise can be effected in practice. It is often necessary to design pumps with a special characteristic to suit certain specified conditions.

RANGE OF APPLICATION OF CENTRIFUGAL PUMPS.

Centrifugal pumps may be used under almost any conditions where it is necessary to deal with liquid or semi-liquid. There is such a wide range of uses that there must necessarily be a considerable variation in design between the various types of pump. Thus, where there is only a small percentage of gritty matter in the liquid to be dealt with, it is only necessary to make certain wearing parts of the pump easily renewable, but when the liquid contains a considerable amount of solid material of an abrasive nature, it is usually necessary to build a pump which is lined either partially or wholly in the interior. There is a good deal of divergence in the design of various makers of dredging pumps, and the designs of dredging pumps by the same maker will also vary according to the nature of the duty to be performed. Sometimes it is only necessary to line the pump at the sides, the volute being unlined, while it is often imperative to fit the whole interior of the pump with liners. There are duties where liners of the best manganese steel become worn through in a few weeks, so that it will be seen how necessary it is for a pump working under these conditions to be totally lined throughout.

INSTALLATION OF CENTRIFUGAL PUMPS.

Centrifugal pumps should be set as near the level of the water pumped from as practicable in order to decrease the suction lift. Suction lines should rise in elevation towards the pump end. In some cases they may be laid level but they should never dip downwards as this may allow air pockets to be formed with consequent unsatisfactory working. There are cases where air pockets are unavoidable, and it is then necessary to keep the highest point of the suction pipe in communication with a condenser or vacuum pump. As a rule it is not advisable to exceed a suction life of 20 ft. including friction, but where it is necessary to go beyond this, the suction pipe line should be tested to a pressure of 30 lbs. per square inch and should be absolutely tight under this pressure. It should be remembered that the effect of air in a centrifugal pump is to reduce the quantity of water delivered. The ultimate result of excessive air leakage is that the water column becomes broken on the suction side and the pump will stop delivering. Where the water flows into the pump under a head, if it is impossible to avoid loops or pockets, it is advisable to place an air cock at the highest point of the pocket to release the accumulated air.

The speed of water in delivery pipes should not exceed about 500 ft. per minute unless the pipe line is very short. It frequently happens that the output of a centrifugal pump with a certain size of delivery or suction branch is much too great for the pipes to be made the same size as the branches. In such cases it is advisable to fit taper pipes next the pump branches. The length to make these taper pipes to give the best results is given by the formula

$$\text{Length} = \frac{D-d}{2 \times 0.12}, \text{ approximately equal to } 4\,(D-d)$$

Where D = diameter of large end in inches
d = diameter of small end in inches
0.12 = constant representing tan. $6°\ 50'$.

This angle of about 7° has been found by experiment to be about the best for water.

Methods of Priming.—Where centrifugal pumps are working on a suction lift it is necessary to charge them and the suction pipe with water. There are two methods of doing this:

(1) To withdraw the air from the pump casing and suction pipe, thus causing a vacuum which becomes filled with water from the suction tank or sump.

(2) To place a foot-valve at the bottom of the suction pipe and fill the pump and suction pipe with water.

As examples of the first method we have the steam ejector, compressed air ejector, and ejector operated by water. Any of these ejectors may be used according to the facilities offered. The ejector is generally placed on the top of the pump casing and when the pump and suction pipe have become filled with water the steam, compressed air, or pressure water is shut off and the centrifugal pump started up. The air in the suction pipe and pump may also be withdrawn by means of a vacuum pump. This is operated by hand or power according to size. In the smaller sizes of centrifugal pump, the air pump is often attached to the pump bedplate but in the larger sizes where power is required it is generally made an independent unit. In the case of intermediate sizes, it is sometimes driven from the centrifugal pump spindle by means of a belt or chain.

In the second method, the foot valve should be placed sufficiently far below the surface of the water to ensure that "solid" water is obtained under all conditions of pumping. Where pumps are charged by means of a foot valve there is another point which it is important to notice. Suppose a pump is running and the driving motor stops suddenly, due to the blowing out of a fuse or from some other cause, then if the pump is fitted with a foot-valve the column of water in the delivery pipe drops down and closes the valve. It can be readily understood that in cases where the static head is considerable, the pressure caused by this column of water being suddenly brought to rest may become very great, and as the pump casing is generally weaker than the pipes on account of its larger diameter, there is great danger of the pump being burst and destroyed. It is, therefore, necessary where the static head (including suction) is greater than 30 or 40 ft., to fit a check valve on the delivery side of the pump. It is often handy to fit this valve with a by-pass which can be opened for the purpose of charging the pump.

Design of Foot-Valve.—The design of the foot-valve selected should be such that the flap can be easily raised by the centrifugal pump, as, if the flap is too heavy, the valve does not come fully open and the pump will not deliver its full quantity. In fact, cases have occurred where the centrifugal pump could not open the valve at all on account of the excessive weight of the flap. Foot-valves should always be chosen so as to give a clear area through them at least equal to the area of the suction pipe. In large sizes foot-valves, where used, are generally of the multiple flap type, though it is usual to employ one of the other methods of charging for large centrifugal pumps. Strainers are nearly always fitted to the suction pipes, and these should always be of large area.

TYPES OF CENTRIFUGAL PUMPS.

Centrifugal pumps may be broadly divided into three different classes, according to the head to be pumped against, and illustrations are given to show variations of these classes. The classes are as follows: (1) Ordinary centrifugal pumps for heads up to about 150 ft.; (2) twin series centrifugal pumps for heads up to about 300 ft.; (3) turbine pumps, i. e., centrifugal pumps having fixed guide-blades in the pump casing and suitable for pumping up to the greatest heads.

In cases where the suction lift would be too great for the pump if placed on the ground level, or where on account of the expense of excavating a larger well, or where on account of risk of flooding it would not be advisable to place the motor down along with the pump, vertical spindle pumps are adopted. Thus, where sewage has to be pumped, the liquid should flow into the pump by gravity, as gas is given off more freely in a partial vacuum, such as occurs in a pump working with a suction lift. In these cases the motor is kept on ground level and the pump spindle is connected to the motor spindle by means of vertical shaft fitted with suitable intermediate guide bearings. The weight of the motor spindle and armature is taken by the motor thrust bearing, and a common way to support the vertical shaft is by means of a thrust bearing contained in the cast iron stool supporting the motor. A flexible coupling is generally fitted between the motor spindle and the vertical shaft and sometimes also between the vertical shaft and the pump spindle. Where this is done it is necessary for the pump to be provided with a bearing to take the weight of the pump spindle and impeller, but where the lower flexible coupling is discarded and solid coupling fitted, the weight of the rotating parts of the pump is taken by the thrust bearing in the motor stool.

In cases where there is a very considerable rise and fall in the level of the suction water, say, greater than 20 to 25 ft., where there is a difficulty in charging the pump, or where for the sake of convenience it is advantageous to do so, the vertical spindle pumps may be totally submerged. In such cases the pump is fitted with bearings of lignum vitae which are lubricated by the water, and any of the intermediate bearings which are likely to become submerged are also fitted with lignum vitae and arranged with a water service for use when these bearings are not submerged. With submerged pumps, the lower flexible coupling is done away with.

With all vertical spindle pumps it is most important that the foundations and [illegible] rigid so as to prevent any possibility of relative movement between the pump and the motor. It is, of course, essential that the shafting be properly aligned. As vertical spindle pumps having shafts up to 80 ft. long have been built it can be readily understood that there is a considerable range for pumps of this type.

Then there is also what is termed a "multirotor" pump. One of these pumps recently built had six impellers all on the same shaft. When driven by an exhaust turbine at a speed of 2,500 r.p.m. it delivered 8,000 gals. per minute on a total head of 25 ft. Each of the six impellers passes its share of the water against 25 ft. head, the water being split up in this manner to enable a high speed to be obtained on low head with a large quantity of water, thus making the turbine drive practicable. The pump casing is horizontally divided so that the whole of the inside of the pump can be inspected by lifting off the top half. Special diffuser pipes are fitted on the suction and delivery in order to maintain a good hydraulic efficiency.

Twin series pumps consist of two centrifugal pump casings bolted together. The first pump delivers into the second and thus for a given speed double the pressure is obtained from a twin series pump as compared with a single pump. In some cases it is found more convenient to have a motor between two single pumps, one at each end of the bedplate, and by suitable arrangement of piping and valves, the pumps can be put either in series or in parallel. This is an excellent arrangement where it is desired to have pumps suitable for either domestic or fire service as required.

The pumping plant supplied for Edmonton, Alberta, is an example of twin series pumps. There are two sets of twin series pumps, each driven by an auto-synchronous motor of 850 b.h.p. at 900 r.p.m. Each pump, when working by itself, gives an output for domestic purposes of 10,000,000 gals. per day (say 7,000 gals. per minute) against a pressure of 110 lbs., and when the two pumps are put in series, the combined output of the plant for fire purposes is 7,000 gals. per minute against a pressure of 160 lbs. The pump casings are fitted with gunmetal liners to prevent corrosion, the pump covers are fitted with impeller bearing rings, the impellers are of phosphor bronze, and the spindles are of nickel steel fitted with gunmetal sleeves.

In ordinary centrifugal and twin series pumps it is most important that the pump volute should be properly designed in order to reduce the speed of the water after it leaves the impeller in such a manner as to get the greatest efficiency from the pump, and, of course, the impeller has to be designed so as to give the water the correct velocity, including the correct direction, both at entry and at exit. In the case of turbine pumps, the water, when it leaves the impellers, is taken charge of by fixed guide vanes. These guide vanes must not only have the correct angle for the water leaving the impeller, but they must also be of sufficient radial depth to ensure that the proper amount of velocity is converted into pressure head. If the guide ring is too short there will be a loss of efficiency due to the water not being sufficiently slowed down before passing from the vanes. If, on the other hand, the guide ring diameter is made very large, the weight of the pump, and hence the price, becomes excessive, and a compromise has to be struck, the practice varying according to the ideas of the respective manufacturers. Guide-vanes are generally made of hard phosphor bronze, or some other similar material to prevent corrosion. When guide vanes become worn, the efficiency of the pump falls and as the vanes introduce an additional element of wear in the pump, where the head is not too great, twin series pumps may be used with advantage, since they have no guide vanes and can be made with practically as good an efficiency as the turbine pumps.

The impellers of multi-stage turbine pumps are generally of the single suction type. The most common ways of taking up the end thrust are by an automatic hydraulic balancing device or by a mechanical thrust bearing. Where the water to be pumped is clean, the hydraulic balancing device works very well, but where the water is gritty a good deal of wear takes place, and it is often found advisable to use the mechanical thrust bearing which very often takes the form of double-acting ball thrust bearing on the end of the pump shaft. Most hydraulic thrust bearings are built on the following lines: A disc which may be termed the piston is attached to the pump shaft inside the casing between the impeller of the last stage and the delivery gland, and this piston has a small chamber behind it. On the side nearest the impeller the disc has a working face corresponding to a similar face on a distance piece attached to the pump casing. If the pressure in the last stage is greater than the pressure behind the piston, the faces tend to move apart and the pressures are equalized and vice versa. This automatic regulation, based on difference of pressure, can, of course, be carried out in a number of different ways, and where the water in the pump is clean it has proved very satisfactory.

An example of this type of pump is the installation at Renfrew, near Ottawa. This is a 12-in. pump with four impellers, and is driven by a water turbine. The pump casing is sheathed with bronze, impellers and guide-vanes are of phosphor bronze, and the spindle is of nickel steel sheathed with gunmetal. The automatic hydraulic balancer is fitted. The flexible coupling is fitted between the pump shaft and the turbine shaft. Some tests taken on this pump, after about 18 months' operation, show that at the required speed of 590 r.p.m., the quantity delivered is fully 10 per cent in excess of that called for by the specification, and the head pumped against 4 lbs. higher than the guaranteed figure.

Turbine pumps driven by a compound enclosed forced lubrication engine through double helical gearing, contained in gear box, are coming into greater favor. A two-stage pump of this kind, built recently for a head of 400 ft., runs at a speed of 1,200 r.p.m. The engine runs at 400 r.p.m., the gear ratio being 1 to 3. This is interesting, as showing how a suitable pump speed may be obtained from a slow-running steam engine, or from a water turbine where low head limits the speed. Well made double helical gears contained in gear boxes have shown themselves to be thoroughly reliable, as indicated by the increasing extent to which they are being taken up for marine propulsion, enabling the turbines and propellers respectively to be run at their most economical speeds, and well designed gears of this type are very quiet. Helical gearing for high-speed machinery does not appear to have been adopted extensively in Canada, but it probably will find considerable favor in the future on account of its convenience and adaptability where it is difficult to work in direct coupled pumps, and particularly in cases where, if direct driven, the low speed of the driving motor would necessitate the supply of a large and costly pump.

Tests of Car Couplers at the University of Illinois.—A series of tests is being conducted by the University of Illinois for the Scullin-Gallagher Iron & Steel Company of St. Louis, on a new style of car coupler which is believed to be an improvement over the coupler now in use. The weak point of most car couplers now in use is the face, this being the place where fracture generally occurs. The new style of coupler eliminates this difficulty by having a solid face and a new type of tail knuckle. A former test made by the University showed that when failure of the new style coupler occurred it was generally at the knuckle. After the first tests, the design was changed to add about 15 lbs. of metal to the knuckle. The new design is now being tested under the supervision of Prof. I. M. Snodgrass of the Railway Engineering Department. Four sets of tests are made: the "striking test," the "jerk test," the "guard-arm test," and the "pulling test." In the striking test the coupler is placed in the drop testing machine and a 1,610-lb. weight let fall on it from varying heights.

Discussion and Data on the Correlation of Water-Borne and Some Other Preventable Diseases.

The so-called Mills-Reincke phenomenon, namely, that where clean water is substituted for polluted water, not only deaths from typhoid fever and other directly water-borne diseases are avoided, but also deaths from other causes including certain of the diseases of the respiratory tract, has been abundantly demonstrated by Sedgwick and McNutt, Hazen, Lederer and others. Some of the evidences of this correlation of water-borne and some other preventable diseases were discussed by C. M. Hilliard, Professor of Sanitary Science at Purdue University in a paper before the Indiana Sanitary and Water Supply Association. Some of the data presented may prove useful to water works engineers and officials in attempting to impress upon cities the importance of pure water.

The diseases that we call water-borne are chiefly those affecting the alimentary tract. Typhoid is the most conspicious and most notorious of these; the dysenteries and infant enteritis are next in this country. In some countries of the world, cholera is extremely important but owing to effective port quarantine service cholera has never seriously invaded this country.

TABLE I—SHOWS DEATH-RATES PER 100,000 IN FIVE-YEAR PERIODS FOR A NUMBER OF CITIES FROM TYPHOID AND TUBERCULOSIS.

City.	Year.	Typhoid.	Tuberculosis.
Berlin	1890-94	8.28	261.1
	1895-99	4.56	216.4
	1900-04	4.02	211.8
	1905-09	4.34	188.2
Hamburg	1890-94	22.11	248.4
	1895-99	6.00	202.0
	1900-04	3.84	178.4
	1905-09	3.60	143.4
Paris	1890-94	25.82	416.0
	1895-99	13.84	384.2
	1900-04	19.04	363.9
	1905-09	10.48	325.8
London	1890-94	14.10	192.2
	1895-99	14.30	174.8
	1900-04	10.36	162.6
	1905-09	4.68	137.3
	1910-12	3.33	131.6
New York	1890-94	21.2	300.0
	1895-99	16.72	260.2
	1900-04	19.12	239.4
	1905-09	14.56	231.0
	1910-12	10.83	199.6
Boston	1890-94	31.50	300.4
	1895-99	32.04	246.0
	1900-04	23.62	225.6
	1905-09	17.64	182.0
	1910-12	9.53	159.6
Chicago	1890-94	96.10	201.8
	1895-99	37.56	179.4
	1900-04	28.78	174.0
	1905-09	16.50	172.8
	1910-12	10.66	168.6
Washington, D. C.	1890-94	88.74	310.6
	1895-99	63.10	288.2
	1900-04	58.74	257.2
	1905-09	39.56	230.0
	1910-12	22.06	209.3

The belief is growing that the Mills-Reincke phenomenon, above defined, is better explained on the hypothesis that the relationship is indirect rather than direct; that water does not contribute disease germs of tuberculosis and bronchitis for example, but that communities with unclean water supplies are generally unclean communities with dirty streets, alleys and houses.

Another indirect factor is that intestinal diseases are frequently followed by other diseases that may result fatally and will thus contribute to swelling the death-roll of the secondary disease. A very important contribution to this hypothesis has recently been made by Dr. Woodruff, who, in an exhaustive study, has demonstrated the tendency of tuberculosis to follow typhoid fever. Within the body of man the tubercle bacillus is almost ubiquitous. It is becoming the prevailing opinion of the medical profession that infection with this disease takes place comparatively early in life and that its fatal termination is generally due to some cause that temporarily upsets the biological adjustment between the body and the germs, permitting a relighting of an old infection. It is this relation that typhoid and possibly other intentinal diseases, bears to tuberculosis. Table I taken from from Dr. Woodruff's article, shows how closely the decline in death rate of the two diseases have paralleled.

In 1910 the typhoid death rate in 50 of the largest American cities was about 25 per 100,000 population. In 1913 the corresponding figure had dropped to 12.81. The lowest rate in 1912 was 2.8, at Cambridge, Mass., and the highest 56, at Memphis, Tenn., for the 50 largest American cities. Ten of the 50 largest cities had rates of 8 or lower.

As to the criterion to be applied in the case of cities of lesser population as regards their normal or sickly condition, it seems as though a higher typhoid rate must be permitted. As the most reliable and most recent source of material for determining this, Professor Hilliard took the New York State Health Bulletin for January, 1914, which reviews the past year's vital statistics for various cities of the state, and combined the tables there given to show the total death rate and the typhoid rate so that we may compare the correlation between them which we have previously assumed would be close.

Certain well defined things may be inferred from Table II. In the first place it will be seen that the high typhoid rate is, as a general thing, accompanied by a high general death rate. The typhoid rates tend to increase as we study the smaller communities. The relation between the kind of water used by a community and the typhoid rate does not seem to be a constant. It is obviously unfair to judge a community below a certain population, say 15,000 by its typhoid rate for a single year as one death makes such a profound difference. Small communities must be studied in groups or over a period of years to yield the information desired. From a study of the data presented and other similar material not here given Prof. Hilliard believes that communities with a population of over 20,000 should at the present time reduce their typhoid rates to 10 or less per 100,000 population providing the water supplies are pure. Six of the 12 cities cited above for all practical purposes have done this. The urban rate for New York State is 9.8. The standards here arbitrarily set are for the present not for the future. As communities, free from gross sources of infection, gradually free themselves from the seeds of disease in the human intestines we may expect more from them.

The rural community is peculiarly difficult to deal with in considering the relation of water to disease as the small figures make general interpretations impossible. Taking New York State again, we learn that the rural typhoid rate (communities with 5,000 or less) is 12.1.

TABLE II—SHOWS CITIES IN NEW YORK STATE, POPULATIONS, DEATH-RATES PER 1,000, TYPHOID DEATH-RATES PER 100,000 AND THE WATER SUPPLIES USED DURING THE YEAR 1913.

Rank.	City.	Population.	Death-rates per 1,000.	Typhoid death-rates per 100,000.	Water supply
1.	Schenectady	86,305	12.2	5.8	Well and spring.
2.	Utica	80,256	19.1	10.0	Streams and reservoirs.
3.	Yonkers	90,156	14.2	12.2	Filtered.
4.	Binghamton	51,300	22.0	15.6	Filtered river.
5.	Troy	77,382	18.2	18.0	Unfilt. stream and reservoir.
1.	New Rochelle	33,461	9.4	3.0	Unfilt. streams and reservoirs.
2.	Amsterdam	34,283	14.3	8.8	Unfilt. streams and reservoirs.
3.	Mt. Vernon	34,066	13.4	8.8	Unfilt. streams and reservoirs.
4.	Poughkeepsie	29,263	16.8	10.2	Filtered river.
5.	Rome	22,172	19.7	22.6	Unfiltered river.
6.	Niagara Falls	34,013	17.5	29.4	Partly filtered river.
7.	Cohoes	24,968	15.5	32.0	Unfiltered river.
1.	Ithaca	15,341	16.2	13.0	Well and spring.
2.	Corning	14,596	12.2	13.7	Well and spring.
3.	Ogdensberg	16,337	25.4	18.4	Unfiltered river.
4.	Glens Falls	16,[illegible]96	17.6	18.6	Unfilt. stream and reservoir.
5.	Middletown	15,570	21.5	38.5	Filtered stream.
1.	Oneonta	10,251	14.8	00.0	Filtered streams.
2.	Rensselaer	10,952	12.6	00.0	Filtered river.
3.	Geneva	13,099	14.2	38.2	Unfiltered lake.
4.	Little Falls	12,886	19.0	38.8	Unfiltered streams.
5.	Plattsberg	12,015	17.9	50.0	Unfiltered streams.
6.	Hudson	12,030	17.8	58.2	Filtered streams.
1.	Oneida	8,568	18.3	11.7	Unfiltered streams.
2.	Port Jervis	9,622	19.6	20.8	Unfiltered streams.

Methods Used in Appraising the Water Works of Honolulu, Hawaii.

II.—RESERVOIRS, PUMPING STATIONS AND MISCELLANEOUS STRUCTURES.

The methods and unit costs used in appraising the Honolulu water works distribution system were described in this journal of date of April 22, 1914. The present article relates to the appraisal of reservoirs, pumping stations and certain miscellaneous structures. The treatment of the subject is confined largely to the computation of depreciation. Some general figures pertaining to the entire works also are given. The information here given is taken from the latest annual report of J. W. Caldwell, Superintendent of Public Works of Honolulu. Mr. A. C. Wheeler, Assistant Superintendent, had charge of the appraisement.

RESERVOIRS.

Fifteen reservoirs are used for water storage in connection with the Honolulu water works. The reservoirs present a great variety of construction for works of this class and they are, for that reason, described here in some detail.

The Punchbowl reservoir was constructed in 1895. The construction is rubble stone masonry sides faced with brick and plaster. The top is of reinforced concrete at the present time, but originally was of wooden frame structure and galvanized iron. The present cover was placed in 1913. The general condition of the reservoir is excellent throughout. A careful examination was made in 1913, when the cover was replaced. The capacity is 1,500,000 gals.

The Kalihi reservoir is of reinforced concrete with wooden frame roof covered with galvanized iron. This reservoir has a capacity of 3,200,000 gals. It was constructed in 1904. At the present time it is in excellent condition. There are a few minor repairs to be made on the roof, but in general, the reservoir is in excellent condition.

The Diamond Head reservoir was constructed in 1898 and has a capacity of 876,000 gals. It is elliptical in shape with over-all dimensions of 125 ft. x 75 ft. x 16 ft. The construction is stone, brick lined, plaster faced with wooden frame roof covered with galvanized iron. The general condition of the cement work is sound, but the roof and screening overhead should have immediate repairs.

The Kaimuki reservoir was constructed in 1889, and the reinforced concrete roof was placed over it in 1911. The construction is concrete for the walls and the roof cover is of reinforced concrete. The capacity is 683,000 gals. The general condition of this reservoir is excellent.

The Makiki reservoir was constructed in 1911, and has a capacity of 759,000 gals. The construction is reinforced concrete throughout, and the general condition is excellent.

The Upper Makiki reservoir is an open reservoir formed by a dam across the bed of the stream. This dam was constructed in 1907. The construction of the dam is stone and concrete. The capacity is 800,000 gals., and the dam is about 80 ft. long and 23 ft. deep at the deepest point. This reservoir has not been in use for some years. It was learned, by trial, that the water was muddy from the reservoir and a wood stave pipe was hurriedly placed to connect the springs with the pipe below the dam. Since that time the dam has not been used. In November, 1913, the reservoir was emptied and cleaned out and work was started to place the dam in use by altering the screen. It is thought by using the reservoir as a settling basin that the water will be fit for use.

The Alewa Heights reservoir was constructed in 1911 and reconstructed in 1913. The original plans called for plastered rock masonry; and it was found that this would not hold water. In 1911 a reinforced concrete slab was placed over the entire inside of the reservoir and a bitumastic solution spread over the concrete. The reservoir is therefore, of rubble masonry and reinforced concrete with a wooden frame roof, having a wood covering. The reservoir is a truncated cone in shape, being 19 ft. in diameter at the bottom, 85 ft. in diameter at the top and 19 ft. in depth. The condition at the present time is excellent.

TABLE I. — STATISTICS OF 39 MUNICIPAL WATER WORKS PLANTS IN THE UNITED STATES, SUPPLIED BY PUMPS, IN CITIES OF POPULATIONS OF 25,000 TO 240,000, AS COMPARED WITH HONOLULU.

Name of city.	Date of report.	Age of plant.	Population	Water consumption. Millions of gals. per year.	Water consumption. Gals. per capita daily.	Head on pumps in feet.	No. of miles of mains.	Feet of main per capita.	No. of services.
Honolulu	Jan. 1, '14	77	[illegible]5,183	5,188	257	32[illegible] 140	1[illegible]5.3	10,049	5,422
Average of 39 cities	[illegible]	33	8[illegible],995	2,924	91	212	133	8.[illegible]	12,760
Maximum	[illegible]	..	2[illegible]9,9[illegible]	8,811	191	[illegible]21	[illegible]93	....	39,389
Minimum	.	.	26,000	[illegible]	26	110	[illegible]		3,326

Name of city.	No. of meters.	Investment. Total.	Investment. Per [illegible] of main.	Investment. Per capita.	Income W. W. Total.	Income W. W. Per tap.	Income W. W. Per capita.	Operating and maintenance expense.	Expense, per cent of revenue.
Honolulu	666	[illegible]	$18,[illegible]	$[illegible]	$[illegible]	$25.[illegible]	$2.[illegible]	$1[illegible]9,314.61	[illegible]
Average of 39 cities	6,8[illegible]	[illegible]	18,[illegible]	2[illegible]	[illegible]	16.00	[illegible]	79,172.[illegible]	[illegible]
Maximum	23,180	[illegible]	[illegible]	60.[illegible]	[illegible],182.[illegible]	31.00	[illegible]	....	.
Minimum	1[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	1.82		.

The Nuuanu No. 1 reservoir was constructed in 1890 by throwing an earth dam across the gulley. This reservoir has a capacity of 21,304,211 gals. and is not covered. The earth dam is entirely of dirt with loose rip rap on the upper side. The condition of this reservoir is good, but some means should be taken for a regular systematic cleaning.

The Nuuanu No. 2 reservoir was constructed in 1890 and has a capacity of 7,301,690 gals. It was formed by throwing an earth dam across a gulley. The dam is entirely of dirt with loose rip rap. This reservoir is open. Its condition is not any too good; there are appearances of seepage below the dam and the boundary lines of the reservoir are choked with vegetation.

The Nuuanu No. 3 reservoir was constructed in 1891 by throwing an earth dam across a natural gulley. The capacity of this reservoir is 10,712,130 gals. The dam is entirely of earth with loose rip rap. The condition is apparently excellent.

TABLE II.—COMPARATIVE STATISTICS FOR THE AVERAGE OF 27 CITIES OF 4,000 TO 47,000 POPULATION AND HONOLULU.

Name of city.	Population.	Miles of pipe.	Tons of pipe per 1,000 population.	Value.	Cost per capita.	Cost per mile of pipe.
Av. 27 cities, 4,000 to 40,000 population....	19,800	42.00	220.00	$500,000.00	$25.10	$11,750.00
Honolulu	55,183	105.03	240.16	938,874.26	17.01	8,939.10

The Nuuanu No. 4 reservoir was completed in 1910 and is formed by throwing a dam across the Upper Nuuanu Valley. The project of constructing this dam had occupied the minds of water works authorities for a long time, and at the culmination of this work it was thought that the water supply for Honolulu had been solved. The capacity of the reservoir, 625,000,000 gals. was ample for many months supply to the city; but up to the present time, the dam has not been filled. The dam was constructed by the sluicing method. Rip rap is hand-placed on the upper side. This rip rap has been found to be an important factor of the dam, since at the time of high water there is sufficient area and length for the waves to form to considerable size, caused by the prevailing wind which is always in the direction of the dam. The cost of this reservoir forms more than half the total cost of all the reservoirs for Honolulu, being slightly less than $300,000. The condition of the dam is excellent.

The Palolo Hill reservoir is under construction at the present time. It has a capacity of 425,000 gals. The reservoir is circular in shape, being 70 ft. in diameter and 14 ft. to overflow. The construction of floor, walls and roof is of reinforced concrete.

The Lower Kaimuki reservoir was constructed in 1898 and had a capacity of 750,000 gals. This reservoir is not in use at the present time, owing to its leaky and decayed condition, and its low elevation. The houses surrounding it are higher, thereby necessitating the new Kaimuki reservoir. The construction of this reservoir was rubble masonry with a concrete face. It was constructed without a cover.

The Pukele reservoir has a capacity of 720,000 gals., and is square in shape. It was constructed in 1878. The construction is rubble masonry with plaster, the walls being supported by an earth fill entirely around it. No cover has ever been placed on this reservoir. Its condition is good, but requires a cover to make it useful.

The Rocky Hill reservoir is of a modern type of construction and is built of reinforced concrete throughout, with a capacity of 726,000 gals.

Depreciation on Reservoirs.—The appraisers assumed that the reservoirs will last indefinitely. This does not mean that all the reservoirs will last forever, but it does imply that the time of their total disintegration is so far removed that depreciation computed on the probable length of life of these structures would likely be erroneous and misleading. Repairs on reservoirs have amounted to almost nothing in the last few years, and it seems reasonable to assume a certain sum which set aside each year would take care of all repairs and depreciation in the reservoirs. It was assumed that $400 per month would be ample to take care of all ordinary wear and tear on reservoirs.

CALCULATION OF DEPRECIATION ON OTHER STRUCTURES AND MACHINERY.

Depreciation on Pumps.—Depreciation on pumps has been assumed as a lump sum. The life of these pumps is indefinite, providing the water works department cares to use them to the end of their life. It is more than likely that as new motive engines come into use the pumps will be replaced at a time prior to their ultimate disintegration from proper use. It is deemed proper, then, to set aside a lump sum each year as a depreciation fund rather than one computed arithmetically on an assumption of a certain length of life.

Depreciation on Boilers.—The life of either return tubular or water tube boilers is entirely dependent on the care taken of them while under steam and the precaution taken to prevent corrosion when the boiler is laid up. Under normal conditions, both types of boiler should last about 25 years, but if subjected to equal abuse, the water tube boiler will show the most rapid deterioration, corrosion soon commencing at the point where the tubes enter the steam drum.

In either type, a complete renewal of the tubes will, in the course of ordinary repair, take place about every seven years, or three times in the whole life of the boiler.

In computing the depreciation on boilers, these facts have been taken into consideration and keeping in mind the amount of time each day that the boiler is used the depreciation has been computed. In two cases a lump sum has been assumed. In these cases the boilers had about reached their estimated length of life. It was considered expedient, therefore, to assume a large depreciation factor, in order to replace these boilers in a very short period of years.

Depreciation on Wells.—Depreciation of cased driven wells has been computed on the theory that wells will last 50 years. This is assumption on our part, since there are no driven wells at that age in Honolulu. There are wells here, however, 19 years old, which apparently have shown no signs of decay. The water seems as free from rust and gravel as when the wells were dug. All computations of depreciation on wells have been computed by assuming the length to be equal to 50 years.

Depreciation on Oil Tanks.—No depreciation has been allowed on oil tanks, due to the fact that there is little reason for depreciation. The oil covering protects the steel and iron from rust, and deterioration, if any, is infinitely small. These tanks store oil for boiler fuel purposes. The oil tank buildings are small in size and value. Lump sum depreciations have been assumed for these buildings.

Depreciation on Venturi Meters.—It has been assumed that venturi meters will last 25 years. While the pipe and underground work would probably last much longer than this, it is believed that the piping and cleaning required and the disintegration which occurs would warrant an assumption of no longer a term of life. All depreciations have been computed on this basis.

GENERAL COMPARATIVE AND LOCAL DATA.

Table I, from Engineering and Contracting of March 8, 1911, gives certain statistics for 39 water plants in the United States with the corresponding figures for Honolulu. The maximum and minimum figures given are averages taken for the 39 cities. Table II shows different items relating to the Honolulu water system compared with the average of 27 cities from 4,000 to 47,000 population, as published in Engineering and Contracting of Oct. 2, 1912. It will be noted that the average population is 19,800, whereas the population of Honolulu is 55,000, therefore giving a comparison which is not as suitable as might be wished. The cost per mile of pipe is considerably lower in Honolulu than in the average of these cities, the difference being $11,750 and $8939.10. The total cost per capita for pipe in Honolulu is $17.01, whereas in the average of the cities it is $25.10. It will also be noted that the tons of pipe per thousand population is about the same.

Bridges

Methods and Costs of Constructing a 450-Ft. Reinforced Concrete Viaduct of the Beam and Girder Type at Fort Worth, Texas.

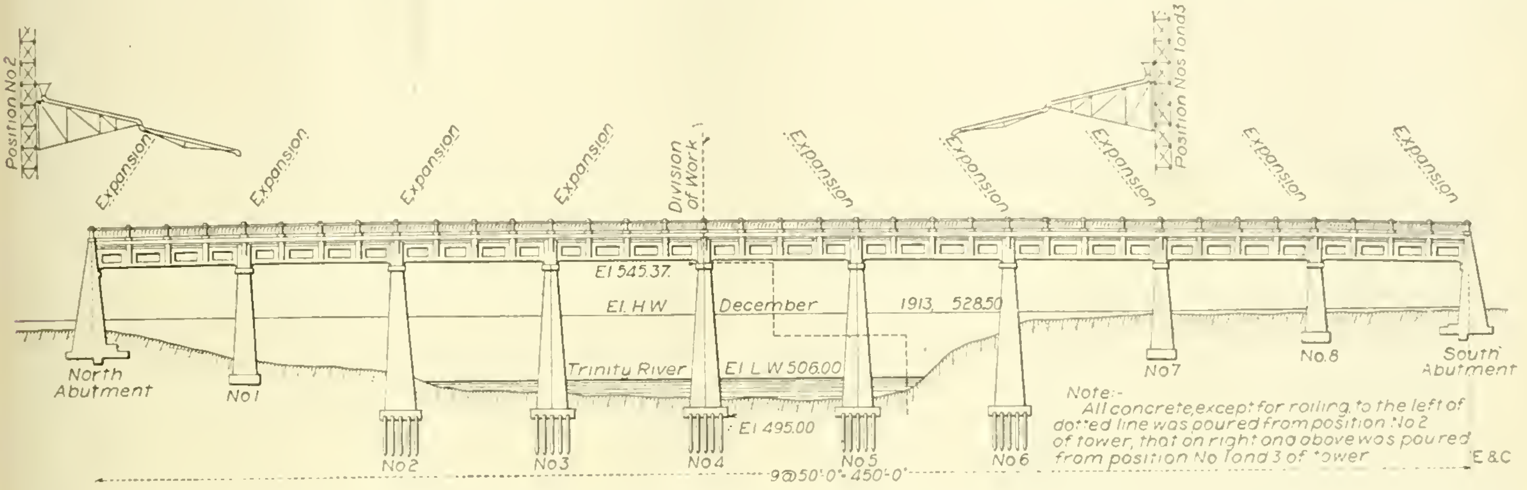
Contributed by E. W. Robinson, Engineer, with
McKenzie-Williams Construction Co.,
Ft. Worth, Tex.

The structure described in the following article consists of nine spans, each 50 ft. center to center of piers. It carries an extension of North Samuels Ave. across the Trinity River, in Fort Worth, Tex. (see Fig. 1) A sufficient number of drawings, together with the loads and allowable stresses used, will be given to illustrate the design features, but the paper will deal mainly with the methods and costs of construction. Construction was started the latter part of May, 1913, and the work was completed in January, 1911. It is one of the four concrete bridges being built by Tarrant County out of a recent $600,000 bond issue, all of which cross the Trinity River within the corporate limits of Fort Worth. During most of the year, especially throughout the summer months, this river is almost dry and is hardly more than a creek, but soon after a heavy rain upstream it becomes a river of considerable size. In one instance the water at the bridge site rose more than 12 ft. in one hour.

LOADS AND ALLOWABLE STRESSES.

The live load used for the floor system consisted of two 50-ton electric cars on each track (the area covered by the four cars being 20 x 100 ft.) and one 21-ton road roller on any part of the roadway (the area covered by the roller being 10 x 50 ft.), together with a uniform load of 100 lbs. per square foot on the sidewalks and remaining roadway surface. The electric cars were double truck, the spacing of the axles of each truck being 5 ft., with trucks spaced 30 ft. on centers. The distance between trucks of adjacent cars was 20 ft. on centers. The car tracks occupy the outside portions of the roadway. The road roller had one-third of its load on the front axles and two-thirds on the rear axle, the axle spacing being 10 ft.

For maximum stresses in the floorbeams, the sidewalks were assumed to be unloaded. The main longitudinal girders were also designed for the above car loading, together with a load of 100 lbs. per square foot on the sidewalks and the remainder of the roadway, the road roller also being considered in connection with the car loading. A load of 200 lbs. per linear foot of each sidewalk was also used to provide for the weight of conduits and pipes. Impact was allowed for according to the formula $I = S \frac{100}{L + 300}$; where I is the stress due to impact, S is the maximum live load stress, and L is the loaded length, in feet, that produces this maximum stress. This formula was used only for the electric cars and uniform roadway loads. Stresses produced by the road roller were increased 25 per cent in all cases. No addition was made for impact to the stresses produced by the uniform sidewalk load. The live load for the piers was not increased for impact.



Note:- All concrete, except for railing, to the left of dotted line was poured from position No 2 of tower, that on right and above was poured from position No 1 and 3 of tower

Fig. 1. Elevation of Reinforced Concrete Viaduct at Fort Worth, Tex., Showing Type of Construction and Different Positions of Concrete Spouting Tower.

The following allowable stresses were used: Tension in reinforcing bars, 20,000 lbs. per square inch; compression in concrete in cross bending, as in beams, 750 lbs. per square inch and in posts, 500 lbs. per square inch; bond, for deformed bars, 120 lbs. per square inch; and shear in concrete, 60 lbs. per square inch. The bearing on the soil at a depth of 6 ft. below the finished surface was taken at 5,000 lbs. per square foot, with an addition for each foot increase of depth over 6 ft. of 100 lbs per square foot. The allowable load on each timber pile was 20,000 lbs.

DESIGN FEATURES.

The floor system is carried by two main girders spaced 20 ft. center to center. These girders support floorbeams every 12.5 ft., which in turn carry the roadway and sidewalk stringers. The width of roadway, face to face of curb, is 30 ft., and each of the two sidewalks has an over all width of 6 ft. Figure 2 (a) shows a half cross-section of the roadway, and Fig. 2 (b) shows a half elevation of a longitudinal girder.

One end of each span is anchored to the pier, and the other end is allowed to slide upon milled steel plates, two plates being used at each girder and stringer bearing. At pier No. 4 both spans are fixed and are reinforced for the negative moment. The expansion and contraction takes place in both directions from this pier toward the abutments. To prevent water from entering the expansion joints 16-oz. sheet copper, bent to a "U" shape, is embedded in the concrete near the top for the full width of the sidewalk and roadway slabs. The specifications called for the use of round reinforcing bars with a mechanical bond, but permitted the substitution of equivalent sizes in square twisted bars. Cast-iron scuppers, spaced 25 ft. center to center, were placed in each curb to provide for the drainage of the roadway. Receptacles for the trolley poles of the street railway, consisting of 3.5-ft. lengths of 9-in. wrought-iron pipe, were embedded at proper intervals in the floorbeams just outside of the curb stringers. The exposed edge of the curb stringers are protected with Wainwright curb bars.

Each pier consists of two plain concrete shafts 4 ft. square under the coping, with a batter of ¾-in. to 1 ft. on all sides. The shafts rest on a reinforced concrete footing which is 3 ft. thick, and are surmounted by a reinforced concrete coping which is 1½ ft. thick and extends 6 ins. on all sides beyond the face of the shafts. The shafts on the land piers are connected at the top by a reinforced concrete diaphram which is 23 ins. thick and either 4 or 10 ft. deep, depending upon the height of the pier above the footing. The shafts of the river piers are connected on both sides, for their full height, by 15-in. reinforced concrete walls, giving the appearance of a solid mass. Figure 3 shows a plan, elevation and cross-section of a typical river pier. A piece of 4-in. vitrified pipe was embedded in the top of the coping over the hollow space, and 6 x 6-in. openings were left in the 15-in. walls near the low-water line to lessen the effect of buoyancy. In addition, the river piers have a solid 90° nosing, or cut-water, on the upstream edge, and each footing is supported by 40 oak piles driven to refusal. The abutments are not reinforced except in the footings and where the wing walls join the abutment proper. Figure 4 shows a plan, elevations and a cross-section of a typical abutment.

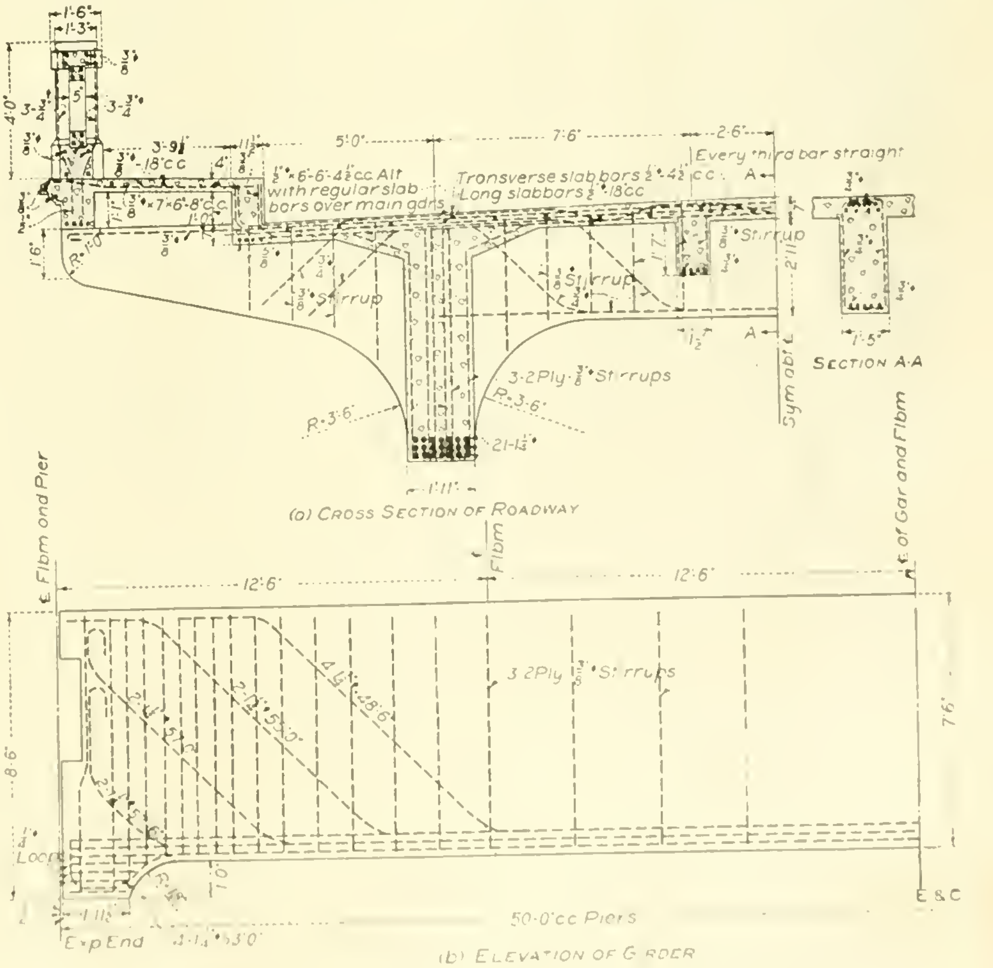


Fig. 2. Cross-Section of Roadway and Elevation of Longitudinal Girder—Fort Worth Viaduct.

The total bidding price of the viaduct was $51,339.63, but owing to increases in the quantities over the engineer's estimate, due to changes in the plans, the final contract price was $57,303.48. Although all of the work never ceased, except during the very heaviest rains, the whole progress was delayed several weeks on account of high water. Some material was lost by floods during September and December. Fortunately, each time the water rose to an unusual height it happened that there were no newly poured spans on the falsework in the channel.

CONTRACTOR'S EQUIPMENT.

The plant used on the job consisted of both new and second-hand machinery, which invoiced at the beginning of the job at $4,852. It consisted of the following: One 5-ton "A"-frame derrick car with a 60-ft. boom, operated by a 7 x 10-in. D. C. D. D. hoisting engine; a concrete chuting plant with an 18-cu. ft. bucket and 121 ft. of steel chutes; a single-drum mine hoist; a 9-cu. ft. gasoline-driven mixer with a self-loader; a 3-cu. ft. gasoline-driven mixer; 25 ft. of swinging leads and a 2,500-lb. drop-hammer which was operated from the boom of the derrick; two 1-cu. yd. turn-over dirt buckets and a 1-cu. yd. clam-shell bucket; and two pumps, one steam-driven and the other gasoline driven. The small mixer was used to mix the concrete used in the railing at the opposite end of the bridge from the main plant, which made it unnecessary to operate the main plant for the small amounts of concrete required for that work. As shown on the general elevation (see Fig. 1) the chuting plant was moved twice, the second move being back to the first location.

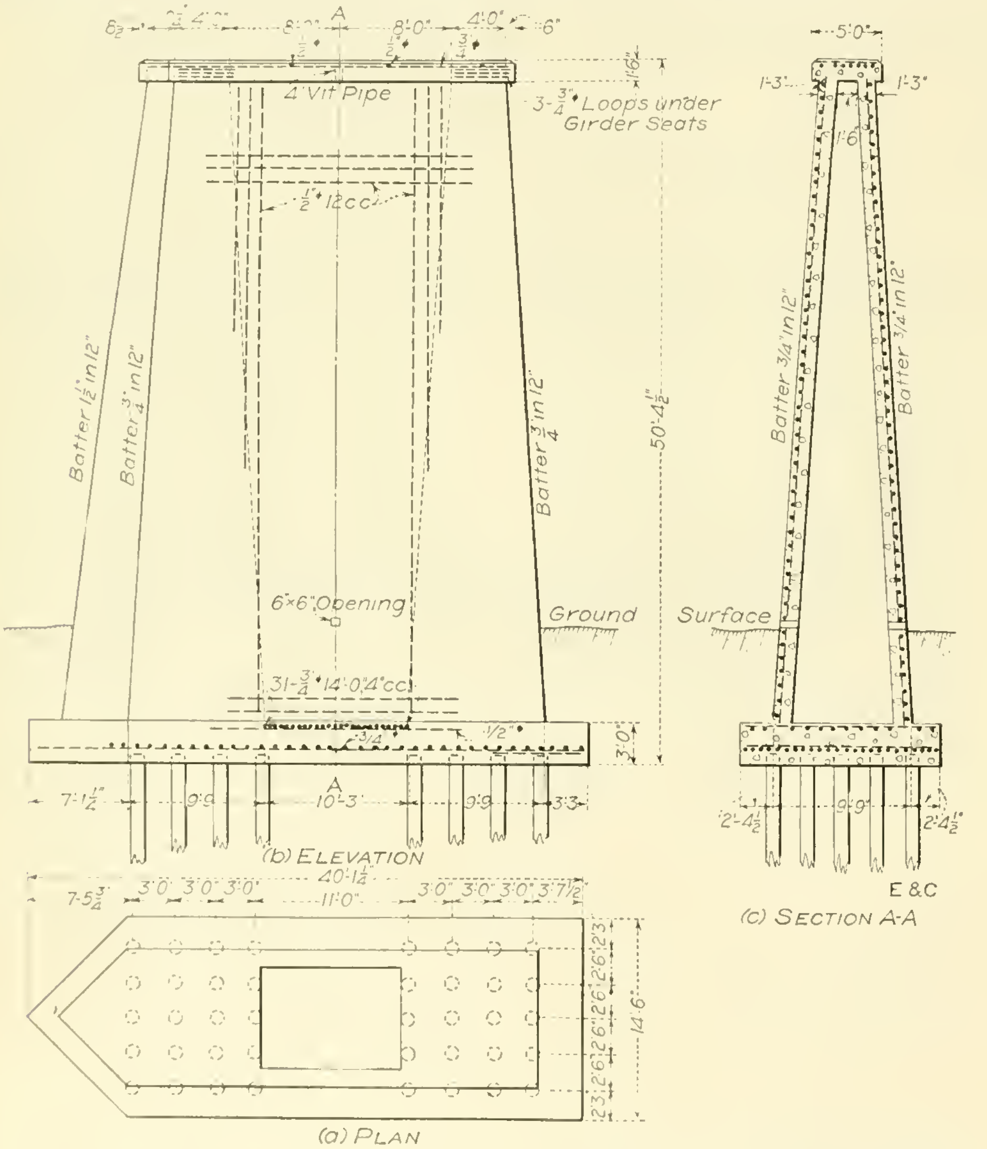


Fig. 3. Plan, Elevation and Cross-Section of Typical River Pier—Fort Worth Viaduct.

In addition to the above mentioned equipment there were the usual petty tools and supplies for a job of this kind, on which depreciation is not far from 100 per cent. The total amount expended for petty tools and repairs for this job was $1,642, and these tools will likely invoice at about $200, showing a depreciation of 87 per cent, which is 5.9 per cent of the pay roll.

EXCAVATION WORK.

Most of the excavation for the land piers was done by slip and wheel scrapers, and was kept trimmed up with pick and shovel. On an average eight to ten teams were employed during the progress of this work. Where convenient, the excavated material was placed back of the abutments to aid in forming the approach fills. The surface soil was black, heavy loam, and when thoroughly dry it was quite hard and would maintain a vertical surface for 15 or 20 ft. without showing any signs of caving.

The excavation for the river piers was mostly done by pick and shovel, being handled in the dirt buckets by the derrick boom. On two piers near the banks of the river a clam-shell bucket was used to advantage until gravel was reached. The piers were carried approximately 12 ft. below the river bed through alternating strata of mud and gravel. The holes were sheeted with 2 x 10-in. lumber, driven by means of a hand maul as the excavation progressed. During the dry months the water was easily handled with a 2 in. duplex steam pump, but when the gravel river bed became more thoroughly saturated after a recent rain it became necessary to install a 2½-in. centrifugal pump in addition to the steam pump.

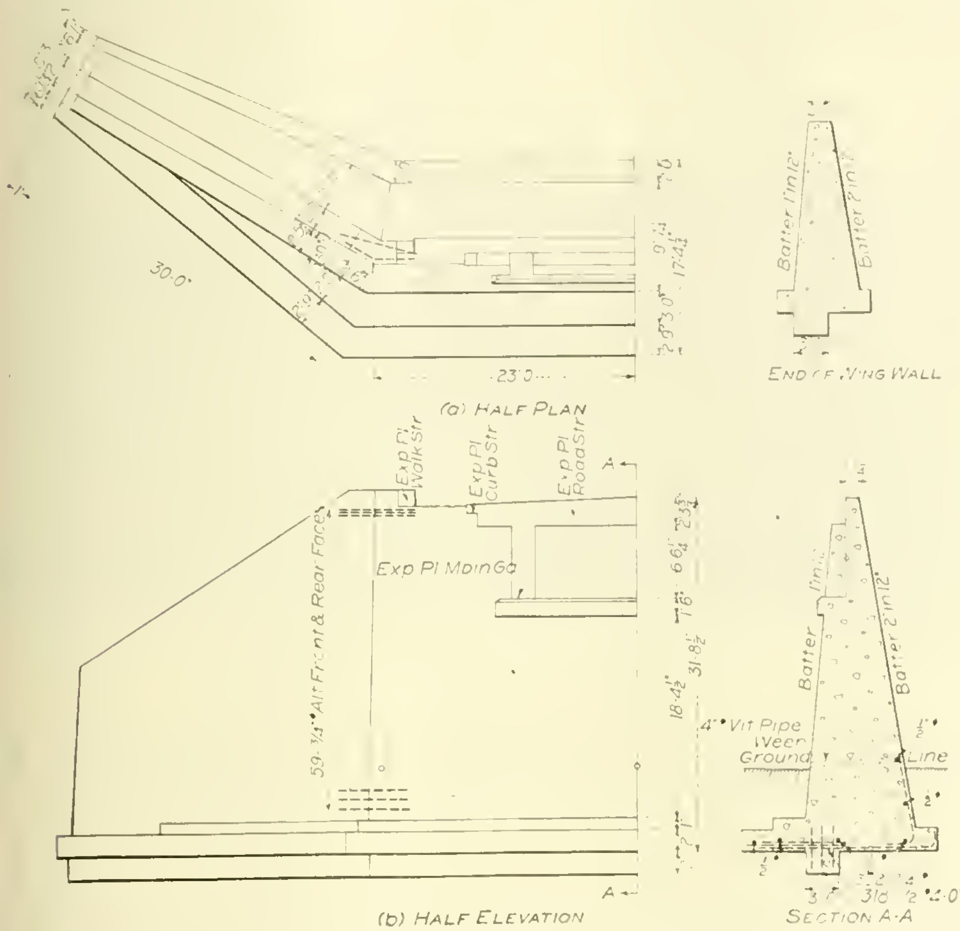
THE FORMS.

The forms for the sides of the piers and abutments were made of 1 x 8-in. and 1 x 10-in. ship-lap, and were supported by 2 x 6-in. studs. The side forms were kept from spreading by ties of No. 8 or No. 3 black, annealed iron wire fastened to 4 x 4-in., 4 x 6-in. or 6 x 6-in. waling pieces on the outside of the studs. The forms for each structure were designed according to the rate of filling which could be expected from the known capacity of the mixing plant. Figure 5 shows a typical portion of the forms for the piers. A sketch of this kind was made on ordinary typewriter paper, and a carbon copy was sent to the superintendent or carpenter foreman in charge, in order that he should not only have proper directions for constructing the forms, but also should know what load they were expected to withstand. The tables of pressures of green concrete on forms and the spacing of studs and wires given in Taylor & Thompson's book on "Concrete Costs" were used, and the writer desires to say that these tables have proved as accurate and reliable as one could expect for data of this kind. The forms held true to line in all cases where the lumber was of average quality and the rate of filling was kept within the limit assumed in the design. In a few cases where a piece of defective timber was used in a place of maximum stress, or where the assumed rate of filling was exceeded, the forms bulged almost to the point of failure. On the pier sides and on the sides of girders and beams, built-up panels were used to advantage in a great many cases, especially on the lower lifts or where the derrick was close enough to handle them. On most of the work, however, it was more economical to build in place the forms for each structure.

The general scheme of design for the form supports of the superstructure is shown in Figs. 6 to 9 inclusive. Figure 6 shows the arrangement and size of the forms and supports under a floorbeam; Fig. 7 shows the forms for the stringers and girders, and the supports between the floorbeams; Fig. 8 shows the forms under the floorbeams and the supports under the roadway stringers; and Fig. 9 gives the size and arrangement of the supports under the girders. The entire floor system for one span, with an assumed construction load of 50 lbs. per square foot, weighed about 333 tons, and was carried on 22 6 x 6-in. posts under the main girders and 72 4 x 4-in. posts under the floorbeams, stringers and slabs. The loads on the small posts varied from 0.9 to 2.4 tons, and on the large posts from 5.5 to 7.5 tons. The footings for the posts of the land spans were designed for a loading of 2 tons per square foot (with no footing less than 1 ft. square). For the main girder posts these footings consisted of a 6-in. layer of concrete, and for the other footings they consisted of a 3-in. piece of oak plank embedded in mortar. At the start there was some question as to the relative economy of using a large number of small posts or a fewer number of large posts capped with timbers. Although the latter offered a small saving in material, this saving was more than offset by the increased cost of erection. Where the size of the job warrants mechanical means of raising the posts and timbers this would perhaps not be true. It was later shown to be a positive advantage to have the load scattered over a large number of footings rather than to have a small number with the loading greatly concentrated. Although the ground was very firm, when dry, it showed a tendency to settle unevenly, and where a large number of posts were used it was possible for one post to settle appreciably and still not affect the structure materially, as the adjacent posts were close enough to take most of the additional load. On another job of similar construction in the same city, where the other method was used, two main girders cracked, presumably due to the settlement of one post under each after the concrete had taken its initial set. On the land spans the settlement varied from ⅜ in. to ⅞ in. This settlement was almost entirely due to the tightening of the joints and to the compression of the wood.

The falsework for each river span was supported on 21 temporary piles, capped with 4 x 12-in., 6 x 12-in. and 8 x 12-in. timbers.

The load upon each pile varied from 6.5 to 15.5 tons. Although the piles were driven a month or more before the falsework was erected upon them, the first river span to be poured had a maximum settlement of 4.5 ins., due to the settlement of some of the piles. However, no settlement took place after the concrete had taken its initial set, and no damage was done other than a slight "belly" in the bottom of the girders. It was afterwards learned that the pile-driver foreman neglected to secure the proper penetration under the last blow of the hammer. As a result of this large settlement the succeeding timbering was reinforced at the points of greatest loads by extra posts set on concrete footings.



(a) Half Plan

(b) Half Elevation

Section A-A

Fig. 4. Plan, Elevation and Cross-Section of Typical Abutment—Forth Worth Viaduct.

One feature of the design of the floor system, which is of special interest to the contractor, was that stock widths of lumber could be used for all beam and girder bottoms, their width being 11.5, 17 or 23 ins. In all cases 2-in. material was used for beam bottoms except under the main girders where the thickness was 3 ins. The sides of the forms for the beams and girders and for the bottoms of the slabs were made of 1-in. shiplap, supported by 2 x 4-in. and 2 x 6-in. studs and joists. In most cases the sides of the beam and girder forms were prevented from spreading by ties of No. 3 or No. 8 black, annealed iron wire. The ties were sometimes made of ½-in. bolts, but the bolts did not prove economical for all locations and conditions.

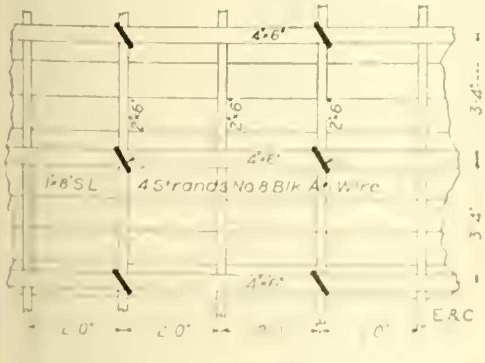


Fig. 5. Typical Section of Forms for Piers and Abutments—Fort Worth Viaduct.

Figure 10 shows a view of the form work in place for the floor system, with the concrete tower in its end position. Figure 11 shows a view of the viaduct, with the tower in its central position, the end portions of the bridge being practically completed.

CONCRETE WORK.

Materials and Proportions.—The concrete for the substructure was mixed in the proportions of 1:2½:5, and for the superstructure, in the proportions of 1:2:4. The top ½ in. of the sidewalks was surfaced with a 1:1 cement mortar, which was floated and troweled to a smooth finish. The sand was bank sand from local pits. It was delivered on the job for $1.20 per cubic yard. The stone used was crushed limestone from a nearby town. It was required to pass a 1-in. ring for the 1:2:4 concrete, and to pass a 2-in. ring for the 1:2½:5 concrete. The stone cost $1.26 per long ton of 2,240 lbs. f. o. b. cars at a team track located about ¼ mile from the work. It was unloaded by contract for $0.25 per long ton. One-man stone cost practically the same on the job as the crushed stone. About 250 tons of this were used in the abutments and pier shafts, but little saving was effected by its use, owing to the high cost of placing it, as the yardage in each place did not warrant much outlay for hoisting, etc.

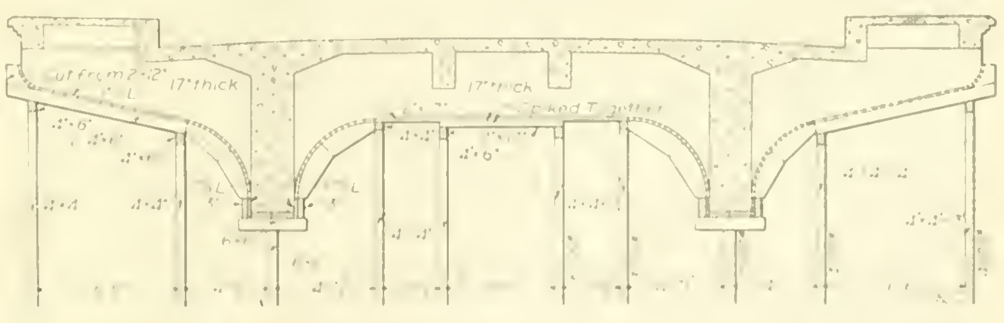


Fig. 6. Forms and Supports Under Floorbeams Used for Constructing Fort Worth Viaduct.

Mixing and Placing.—All of the concrete, except a small yardage in a part of the railing, was mixed and placed through the central plant at the tower. The whole superstructure between curbs for one span was poured in one continuous operation, the sidewalk stringer, the curb stringer and the sidewalk slab being poured a few days later. The reason for this was because of the difficulty of supporting the side forms for the curb stringers to line and grade while the concrete on the roadway slab was still green.

Provision was made for a street railway track over each main girder by embedding in the concrete slab two rows of 1-in. machine bolts for each rail, spaced 2 ft. 3 ins. on centers. Considerable difficulty was experienced at first in holding these bolts to line and grade. The first method used was to make and set wooden boxes for each pair of bolts, the boxes being removed and the bolts concreted in place after the other concrete had set. Owing to the closeness of the spacing of the slab bars, however, this was found to be a slow and expensive procedure. The method finally decided upon was to use these boxes spaced every 10 or 12 ft. to support an inverted T-beam which was made of two 2 x 6-in. pieces nailed together and on which the bolts were hung to line and grade. This not only held the rail bolts to their proper places, but also provided supports for the movable wooden chutes through which the concrete was deposited to all parts of the span. It also furnished a platform from which the men could work. The steel chutes from the tower led to the center of the span where they would not have to be shifted, and the concrete was directed from there by means of short pieces of wooden chutes, which could be easily handled by two men.

The main towers, which were approximately 110 ft. high, were so designed that the concrete would reach all points on a grade of not less than 1 in 4. However, the arrangement was not always followed as intended, and occasionally this slope was flattened. The maximum distance which the concrete was chuted was 250 ft., and no trouble was experienced except when a dry batch was run through, which caused the following batch to spill over the sides of the chutes.

The stone and sand were dumped as near as possible to the mixer, and were conveyed from the piles to the hopper of the self-loader in wheelbarrows. Although at times of short duration the concrete was properly mixed and deposited at the rate of 30 or 35 cu. yds. per hour the rate could not be maintained for any great length of time. A good average for the whole day's run was 20 cu. yds. per hour. A high-speed mine hoist was used for raising the bucket in the tower, and there were no delays from that source. The typical organization of the concrete gang for a day's run was 7 men on the sand, 6 men on the stone, 2 men bringing and emptying the cemment into the mixer, 1 man each to run the mixer and hoister, and 4 men on top placing and working the concrete around the reinforcement and shifting the chutes. However, the above organization would vary according to the location and condition of the stone and sand piles.

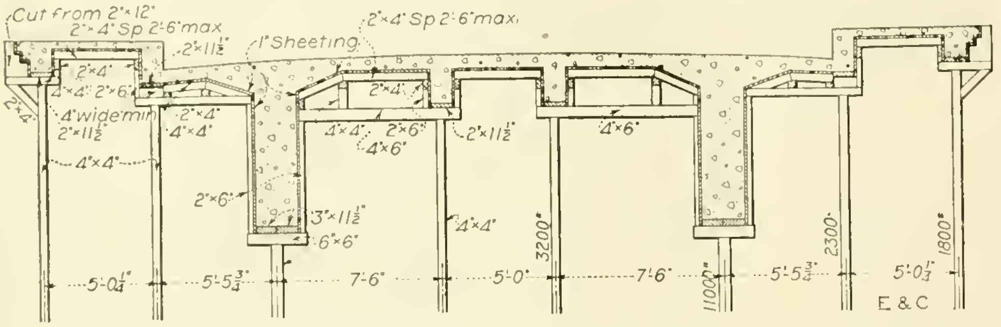


Fig. 7. Forms for Stringers and Girders and Form Supports Between Floorbeams — Fort Worth Viaduct.

CONCRETE RAILING.

The railing was constructed in three separate operations. The 10 x 12-in. foot rail was first cast in place, leaving an opening at each post about 8 ins. less than the width of the post. The panel and the hand rail were each cast separately nearby on the bridge floor, and were allowed to cure thoroughly before being assembled. The panels were cast long enough to project 4 ins. into each post, and wide enough to fit into a 1¼-in. groove in the top of the foot rail and in the bottom of the hand rail. As soon as the separate pieces were assembled the post forms were bolted together in place, and the posts were poured about the ends of the panels, foot rails and hand rails and around the ¾-in. rods which projected down into the sidewalk stringer. As soon as the post forms were removed all the joints were pointed, and when the concrete was properly seasoned the surfaces were finished by painting with grout and rubbing down with a carborundum brick. This method removed small defects, filled small holes, and gave the concrete a uniform and pleasing texture. An 18 x 24-in. bronze name plate was embedded on the inside face of the right hand post at each end of the bridge.

Wood forms were used for all of the railing with the exception of the semi-circles at the top of the 6-in. openings in the panels; these were made of heavy galvanized iron. The bed or pallet for the panels was made of tongue-and-groove flooring nailed to 2x6-in. joists. As all openings in the assembled panels were to be vertical, regardless of the grade of the bridge which varied from level to 2 per cent, it was necessary to shift the position of the cores for every change of grade. After pouring the panel the cores were withdrawn as soon as the cement had taken its initial set and would stand vertical. This gave an opportunity for dressing up a great many air holes and other imperfections before the concrete had set hard.

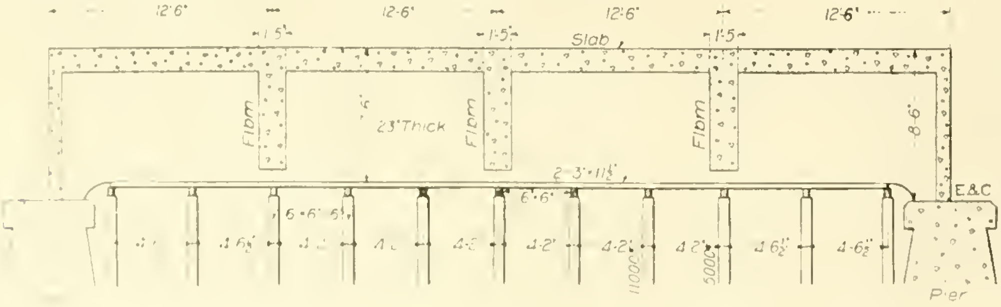
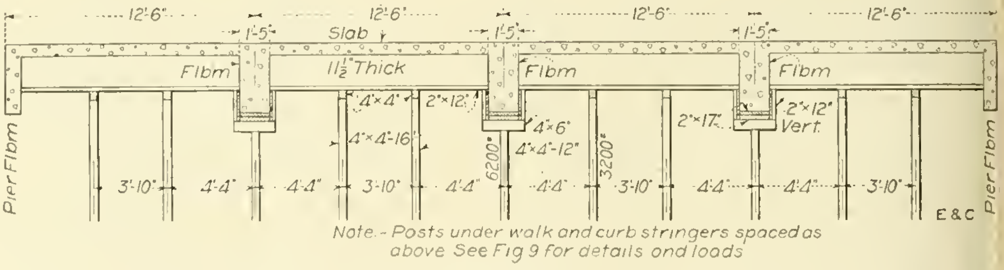


Fig. 9. Forms and Form Supports Under Girders of Fort Worth Viaduct.

An expansion joint was provided in the railing, over the joint in the floor system, by wrapping both the ends of the panel and the hand and foot rails with tar paper before pouring the post. This allowed the same movement in the railing as in the span proper. No anchorage was provided between the foot rail and the walk on which it rests, except through the posts.

The hand and foot rails are each reinforced with four ⅜-in. square twisted bars. The panels are reinforced with eight longitudinal bars and two vertical loops around each opening, all of ⅜-in. square twisted rods. Each post is reinforced with six ¾-in. square twisted bars, running from the top down into the walk stringer, and four ¼-in. plain round hoops equally spaced. In addition to this reinforcement each post which carries a lamp post has two 1-in. conduit pipes embedded in its center, together with four ¾-in. anchor bolts.



Note.- Posts under walk and curb stringers spaced as above See Fig 9 for details and loads

Fig. 8. Forms for Floorbeams and Form Supports Under Stringers — Forth Worth Viaduct.

COST DATA.

The general foreman, or timekeeper, was required to make out daily reports showing the number of hours spent each day on each item of work, together with the wage rate. These reports were filed in the office, together with the progress charts and photographs, and constitute a complete record of the progress of the work as well as furnishing a method of determining the cost of the various classes of work done. The man who made out these reports was required to make the totals check with the total time turned in for the pay-roll. In this way the total cost of labor is absolutely correct, although the different items may be in error to some extent. It is believed, however, that none of the items are very far from correct; because the man making them out had an incentive to make every one as low as possible, and he would naturally try to get the proper distribution. The incentive in this case was due to the fact that his cost data were compared each month with those from another job which was going on at the same time and which was almost identical with this one.

The items to which the time was charged on the daily reports were as follows: General, Dry Excavation, Wet Excavation, Erecting and Wrecking Substructure Forms, Erecting and Wrecking Superstructure Forms, Reinforcing Steel, Rigging Pile Driver, Driving Temporary and Foundation Piling, Preparing Concrete Plant, Mixing and Placing Concrete, Railing, Finishing, Grading, and Miscellaneous. The item "General" is intended to cover all labor which is general in its nature and cannot be charged to any particular class of work, such as that of the superintendent, general foreman, night watchman, and water boy. This cost is kept as a separate item, and is distributed to all other items in proportion to their total costs. The term "Miscellaneous" includes those odd jobs which are not of sufficient importance to warrant making separate record of them, together with the cost of un-related and unexpected work, such as is caused by floods, etc. There is a temptation for the man who makes out the reports to crowd this item to keep the other costs low, and it is therefore necessary to watch this item more than all the rest. This was accomplished by requiring that the foreman write on the report what was actually done during the time charged to "Miscellaneous."

The item "Rigging" was separated to giv an idea of what such work costs, and wa pro-rated at the end of the job between pil driving and the wet excavation. In the sam way "Preparing Plant" was kept separatel simply to determine what this item amounte to for a job of this kind, and was added t

TABLE I—QUANTITIES AND UNIT COST OF VARIOUS ITEMS.

Item and quantity.	Unit cost
Dry excavation, 1,819 cu. yds.	$0.38
Wet excavation, 920.6 cu. yds.	2.19
Erecting substructure forms, 38,876 sq. ft.	0.08
Erecting substructure forms, 3,413.6 cu. yds.	0.97
Wrecking substructure forms, 38,876 sq. ft.	0.01
Wrecking substructure forms, 3,413.6 cu. yds.	0.19
Erecting superstructure forms, 44,460 sq. ft.	0.14
Erecting superstructure forms, 1,237.2 cu. yds.	5.34
Wrecking superstructure forms, 44,460 sq. ft.	0.02
Wrecking superstructure forms, 1,237.2 cu. yds.	0.92
Bending and placing reinforcing steel, 123.9 tons	13.42
Driving foundation piles, 200	2.99
Preparing concrete plant, 4,650.8 cu. yds.	0.30
Mixing and placing concrete, 4,650.8 cu. yds.	0.82
Railing, complete, 906 lin. ft.	1.39
Railing, complete, 76.9 cu. yds.	16.44
Placing rip-rap, 690 cu. yds.	0.12

MISCELLANEOUS DATA.

The following quantities of lumber wer delivered and used on the job:

Size.	Ft. B.
1-in.	60,16
2-in. and 3-in.	98,84
4x4-in. to 8x12-in.	37,84
Miscellaneous and second-hand	20,80
Total, various sizes	217,64

"Mixing and Placing Concrete" to get the true cost of this important item. The cost of driving temporary piling was added to the cost of erecting the superstructure to get the actual cost of the forms.

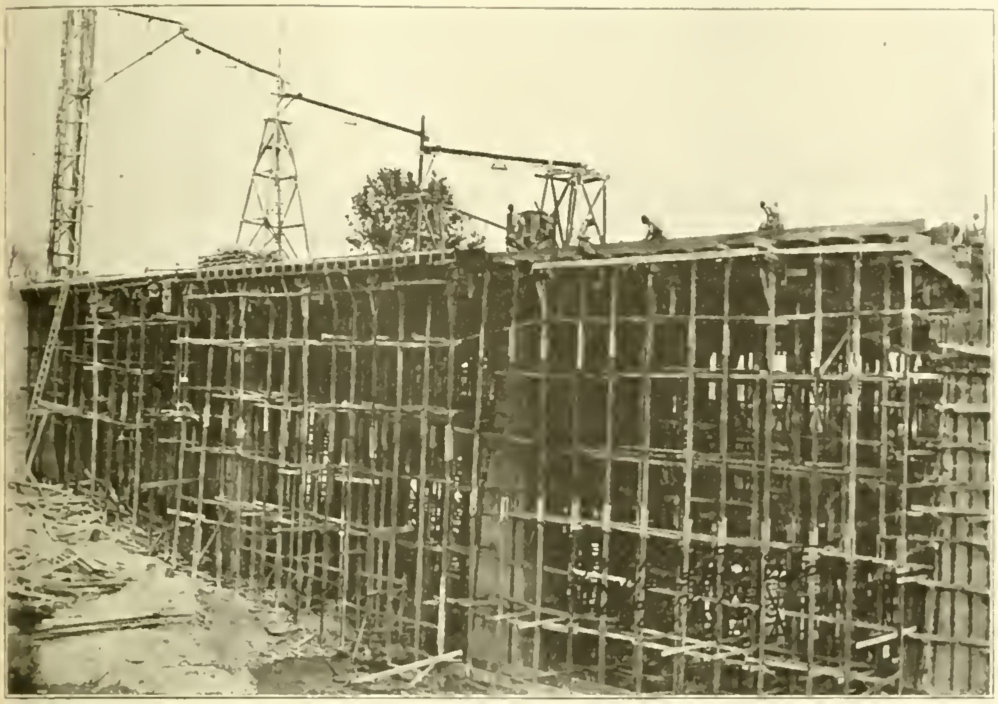


Fig. 10. View of Viaduct at Fort Worth Showing Forms and Supports for Floor System in Place—Tower in End Position.

Eight hours constituted a day's work except in an emergency. For the first week or two common laborers were paid $1.75 per day, but for practically the whole work these laborers were paid $0.25 per hour. For the last month or two the price paid to the common laborers was cut to $0.20 per hour, with the exception of a few of the more energetic ones. Colored labor was used largely throughout the job, and proved to be fairly efficient, with competent supervision. Carpenters were paid the union scale of $0.50 per hour, with time and a half for overtime and with Saturday afternoons off. Skilled laborers, such as riggers and hoisting engineers, were paid from $0.35 to $0.50 per hour. Foremen were paid from $0.50 per hour to $25 per week straight time. The average price per hour for all labor, including general labor, on the whole job was $0.31 per hour. The item "General" amounted to 13.4 per cent of the total labor cost, and it has been apportioned to the different items to obtain the unit costs given in Table 1.

Of this amount between 7,000 and 8,000 ft. B. M. of the 1-in. lumber had a salvage value of from one-half to two-thirds its cost price. There was also about the same amount of 2-in. and 3-in lumber which could be used again or sold as second-hand lumber. Practically all of the large timbers were in good shape at the end of the job, but in the smaller sizes the loss approximated 50 per cent or more.

The quantities of materials given below are reduced to the amounts used per cubic yard of concrete or per square foot of forms, as the case may be:

Nails, 0.101 lbs. per sq. ft. of forms.
Wire, 0.107 lbs. per sq. ft. of forms.
Cement, 1.207 bbls. per cu. yd. of concrete.
Sand, 0.504 cu. yds. per cu. yd. of concrete.
Crushed stone, 1.04 long tons per cubic yard of concrete.

The wire used is reduced to the amount per square foot for the total area formed, although it should be borne in mind that no wire was used on the bottom area of all slabs and beams.

The daily record of the quantity of cement used in all the different members showed that 88.9 per cent of the total amount delivered on the job was used and that 11.1 per cent was lost, wasted or otherwise not accounted for. The quantity of cement per cubic yard of concrete given above includes this waste, or loss, which was added to the amounts taken from the record. The number of empty sacks for which no credit was given at the mill, due to loss and damage, amounted to 8.3 per cent of the total number ordered.

In determining the quantity of sand and stone used per cubic yard of concrete, deduction was made from the total yardage for the actual volume of one-man stone used therein. No record was kept of the quantity of sand and stone used in the separate members.

PERSONNEL.

The viaduct described in this article was designed and its construction was supervised by Brenneke & Fay, St. Louis, Mo. It was constructed by the McKenzie-Williams Construction Co., engineers and contractors, with Mr. A. J. McKenzie as general manager in charge of the work.

Disintegration of Concrete and Corrosion of Reinforcing Metal.

There has been considerable written concerning the disintegration of concrete and the corrosion of reinforcing metal, yet it is difficult for an engineer or contractor to obtain a clear conception of the subject from the published discussions. The Committee on Masonry of the American Railway Engineering Association, in an endeavor to present in a concise form recognized good practice relating to those particular features of concrete construction which must be observed in order to prevent disintegration of concrete and corrosion of reinforcing metal, presented the following report:

The subject of the disintegration of concrete, when investigated with the object of preparing a report that shall be of value to engineers, leads directly to the study of the cause of disintegration and the means whereby it may be prevented. The corrosion of reinforcing metal confines investigation to a study of reinforced concrete. Inasmuch as corrosion of reinforcing metal ultimately leads to disruption of the surrounding concrete and ordinarily presupposes disintegration of the concrete, the subjects are closely allied. It is to be noted that in work where extraordinary provisions are made for peculiar conditions, the requirements of good design and good engineering in general, covering both materials

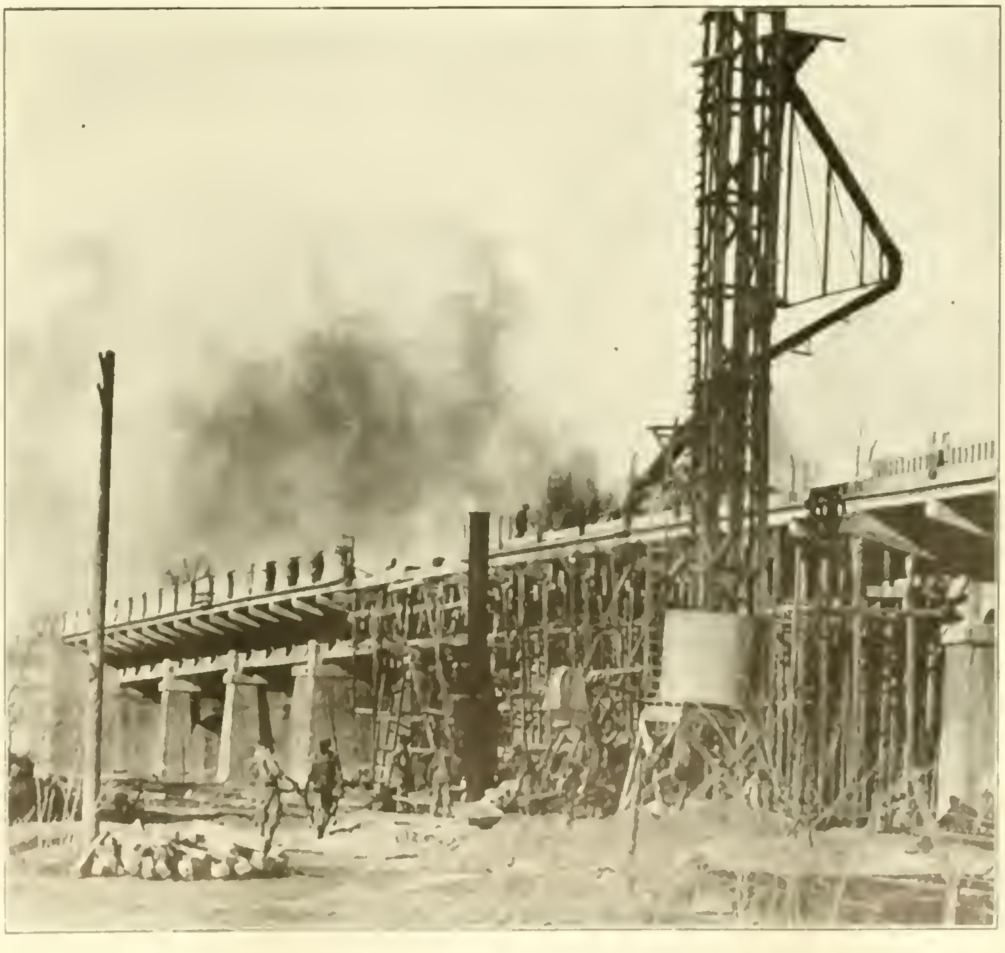


Fig. 11. View of Viaduct at Fort Worth Showing Tower in Central Position—End Portions of Viaduct Are Practically Completed.

and methods, must be none the less rigidly adhered to.

Failure to provide for shrinkage of concrete due to hardening in air, and for expansion and contraction due to temperature changes, is a common cause of cracks which are sometimes ascribed to disintegration.

The requirements for good materials have been described in numerous tests, specifications and reports, together with the methods of determining whether the materials meet the requirements. However, the proper precautions are not usually observed in selecting concrete aggregates. The use of crushed stone screenings as fine aggregate is a frequent cause of disintegration and resultant corrosion of reinforcement.

To obtain good work, competent and ample supervision is absolutely necessary.

CONCRETE IN SEA WATER.

Investigations concerning the effect of sea water on concrete immersed for periods up to 50 years or more; of the relative merits of standard Portland cement and Portland cement made with different proportions of its principal constituents, in resisting the disintegrating effect of sea water; of the effect of varying the proportions of cement in the mortar and concrete; of differently graded aggregates; of the addition of various finely ground materials to the cement after burning; of the relative durability of concrete cast in place as compared with concrete blocks allowed to harden before placing in the sea; and of the effect of various materials added to the concrete mixture to produce impermeability and consequent increased durability, have been made in European countries and in America.

Regarding the chemical composition of the cement, the following conclusions are presented:

Cement containing up to 2½ per cent of SO_3 resists the action of sea water fully as well as cement with lower SO_3 content.

While all the hydraulic cements now in use are liable to decomposition in sea water, Portland cement is the one to be preferred in every respect.

High iron Portland cement and puzzolan cement have failed to show superiority over standard Portland cement in resisting the disintegrating effect of sea water.

Regarding the effect of varying the proportion of cement in the mortar and concrete, in general the rich mixtures have been found to offer better resistance to the attack of sea water. Proportions recommended for mortars are those with 1 part cement to 1 part sand up to 1 part cement to 2 parts sand. The bad condition of mortar leaner than the above after exposure in sea water stands out prominently. In the use of reinforced concrete for maritime works, it is advisable to employ larger proportions of cement than are usual for similar works in fresh water.

Concerning the addition of finely ground material to the cement after burning, it has been found that the addition of puzzolana to Portland cement increases the resistance of the resulting mortar or concrete to the disintegrating effect of sea water.

Regarding the use of any material added to the concrete mixture in small quantities in order to reduce permeability, no results of practical working tests have demonstrated that the effect of any material in reducing permeability is other than mechanical, i. e., to supply a deficiency in fine material in a poorly graded concrete mixture.

Allowing the concrete to harden under favorable conditions before exposure to the action of sea water greatly increases its resistance to attack by the sea water and is recommended wherever possible.

When concrete is deposited under sea water, such precaution should be observed as will prevent the washing of the cement from the mixture.

Forms should be so tight as to prevent the entrance of sea water after depositing the concrete, in order that a smooth dense surface may be obtained.

The combined effect of freezing and of sea water is noted on marine structures in northern latitudes between high and low tide levels. Under these conditions the disintegrating effects are particularly severe.

Dense, properly hardened concrete is not affected by the action of sea water. Where the concrete is porous, however, it is likely to be damaged by frost action, especially between tides. There is no evidence, however, that porous concrete is damaged by sea water in latitudes where there is no frost.

The making of a dense, impermeable concrete by the use of a well-graded aggregate, rich mixture, proper consistency, and good workmanship, and allowing the concrete to harden under favorable conditions before being exposed to the action of sea water, is generally conceded to be an efficient means of satisfactorily insuring the preservation of concrete in maritime works.

CONCRETE SUBJECTED TO THE ACTION OF WATER CONTAINING ALKALIES.

Investigations concerning the effect of ground waters which contain alkalies on concrete have disclosed several instances of apparent disintegration. The following points have been demonstrated in regard to the resistance of concrete to these agencies.

Concrete in which poor aggregates and lean mixtures have been used and in which the material has been carelessly placed when coming in contact with alkali seepage may be affected thereby.

The aggregate should be composed of materials inert to alkalies present in the water. A chemical examination of the sand from country known to contain alkaline soils is recommended.

Water containing substances known to react with the elements of the cement should be kept from coming in contact with concrete until the latter has thoroughly hardened.

Care should be taken to provide a smooth surface and sufficient slope to the extrados of the arch of tunnel linings when the ground water level lies below the tunnel grade to facilitate the flow of seepage water to the sides. The back filling over the arch should consist of porous material such as coarse, crushed stone, for the same reason. Side-drains should be used where necessary and connected with an under-drain, which should be provided in all cases.

The alkalies which are most active in causing disintegration of concrete when allowed to penetrate into the interior of the mass are the sulphates of sodium and magnesium.

The measures to be used in making concrete which is to be exposed to the action of these deteriorating agencies in order to prevent disintegration are the same as recommended for sea water construction. Impermeability is the prime requisite, and the results of experiments and practical tests indicate that concrete, carefully prepared, is just as resistant, if not more so, than if mixed with foreign materials or special preparations.

MISCELLANEOUS CAUSES OF DISINTEGRATION.

Cinders give unsatisfactory results in concrete, especially if there is much coke or porous material present. Such cinders may be improved, if allowed to weather, with occasional washing until the ferrous iron and sulphur have been oxidized and leached out.

Cinder concrete in roofing slabs exposed to the action of locomotive gases is not an efficient protection for reinforcing metal, which has been found to corrode and cause the disintegration of the slab.

Freshly made concrete surfaces in contact with smoke gases at temperatures below 45° F. have failed to harden properly, and experiments indicate that under such conditions the cement is acted upon by the gases. It has therefore been recommended that when heating is done by means of open fires, higher temperatures should be maintained.

EFFECT OF ELECTRIC CURRENTS.

Laboratory experiments furnish most of the information which exists concerning the effect of electric currents on concrete and reinforcing metal. The discrepancy between the conditions in these experiments and field conditions seems to be greater than is the case in other laboratory tests on structural materials, and the information obtained up to this time is difficult of application to field conditions.

It has not been shown that plain concrete is affected by the passage of an electric current through it. (For data on this subject see Engineering and Contracting of March 4, 1914.—Editors.)

CORROSION OF REINFORCING METAL.

Tests and experience have proved that steel embedded in dense concrete will not corrode when located either above or below fresh or sea water level. Concrete, in order to be an efficient protection to steel, must be rich in cement and mixed to such a consistency as to flow around and completely coat the reinforcing metal.

Steel to be embedded in concrete should not be painted.

CONCLUSIONS.

(1) Concrete exposed to the action of sea water, or in contact with alkali waters, or exposed to gases containing sulphur, or in which reinforcing metal is embedded, should be dense, rich in Portland cement and allowed to harden under favorable conditions before exposure to the conditions stated.

(2) Concrete in contact with alkali waters should be made with aggregates inert to the alkalies in the water.

(3) Cinders should not be used for concrete in which reinforcing metal is embedded.

(4) Reinforcing metal should not be painted, but should be thoroughly covered and protected with concrete when in place.

Equipment and Methods Used for Constructing the Fallsway Viaduct, Baltimore, Md.

The Fallsway Viaduct, Baltimore, Md., has a length of about 1,500 ft., and extends from Madison St. to Biddle St. on an average grade of 3 per cent. The viaduct is of reinforced concrete construction, and it occupies the space above the Jones Falls conduits, thus creating a new street.

The lower end of the viaduct consists of an earth fill between concrete retaining walls. The remainder of the structure is a series of 17 sets of concrete arch spans, carrying a reinforced concrete floor system. There are nine arch ribs in each set, spanning the space between transverse girders, which in turn rest on piers continued up from the walls of the conduit (see Fig. 1). The following data, which deal chiefly with the contractor's equipment and the methods used in constructing the viaduct, were taken from a paper, by Mr. Louis R. Gons, in the Cornell Civil Engineer.

DESIGN FEATURES.

The main girders are 3 ft. wide by 6½ ft. deep, and are reinforced with 1¼-in. straight and trussed bars. These girders are spaced 64 ft. apart and are supported by five piers, each 10 ft. long by 2½ to 3 ft. wide (see Fig. 1), making the span between piers about 19 ft. There are nine arch ribs in each span, four of which spring from the mid points between piers of this girder. The skewbacks for these arches protrude the same distance from the center line of the girder as do those for the piers, making it possible to use the same size of centering for all the arches in a set.

The arches are 2 ft. 6 ins. deep at the crown and 3 ft. 6 ins. deep at the haunches. They form a circular arc, with a rise of 1 in 7.8. The three central arches are 3 ft. wide and are reinforced with four 1-in. square bars, placed 3 ins. from the upper surface, and four 1-in. square bars placed 3 ins. from the lower surface. The other inside arches are 2 ft. 6 ins. wide, the outside arches are 2 ft. 8 ins. wide, and all are reinforced with three 1-in. square bars near the upper face and three 1-in. bars near the lower face. Two extra bars are placed in each arch from the springing line to the quarter points. Near the center of the span pockets are made in the arches to contain the transverse floorbeams of the roadway, while spandrel posts are used to support the remaining floorbeams.

The floor slab is provided with expansion joints over each girder, so that the length of a solid slab is never greater than 64 ft. An expansion joint is also placed between the outer spandrel walls and the adjacent piers to permit the free rise and fall of the arches. The joint extends from the top of the arch to the top of the parapet.

A 1:2:4 concrete was used, gravel being used for the interior construction and crushed stone for the outside work—the outside work being later dressed. There are about 11,000 cu. yds. of concrete and 800 tons of steel reinforcement in this structure. The contract price for the outside concrete, for which broken stone was used, was $9.25 per cubic yard; and for the inside concrete, in which gravel was used, $8.90 per cubic yard. The total contract price of the viaduct was $190,693.15.

CONSTRUCTION PLANT AND METHODS USED.

The construction plant is almost centrally located with respect to the viaduct. It consists of a mixing plant at the foot of a 205-ft. tower, situated on the north side of Eager St. The surface of Eager St. at this place is carried on a stone arch bridge, which is about 25 ft. above the bottom of the tower. The material bins are placed immediately alongside this street and between it and the tower, and are built in three compartments, one of 50-cu. yd. capacity for gravel, one of 25-cu. yd. capacity for crushed stone, and one of 30-cu. yd. capacity for sand. The tops of the bins are about 2 ft. above the surface of the street, and two runways extend over these tops, and run down on a short incline over the sidewalk to the street, to provide for filling these bins with sand and gravel, which is hauled by motor trucks. Each bin has a sliding door on the bottom which allows the material to drop into a measuring box, from which it discharges directly into the mixer. Underneath Eager St., on the same level as the tops of the conduits and parallel to them, is a private siding for the unloading of crushed stone and other materials transported by rail. The stone is lifted into the bins by a bucket elevator of 250 tons capacity per day.

The cement house is situated on the north side of the tower and alongside this private siding, so that the cement can be unloaded directly from the cars into this house, which has a capacity of eight carloads. The cement is wheeled by hand a distance of 40 ft. to the measuring box.

The concrete is discharged from the mixer into a $\frac{1}{2}$-cu. yd. bucket, which is then hoisted up the 205-ft. timber tower, and is automatically dumped into a hopper, when it reaches the top. The boiler plant used for operating the hoist and mixer is provided with two boilers to insure continuous operation.

The tower is built of four 6x6-in. vertical posts of 11-ft. lengths, braced horizontally and diagonally with 2x6-in. members. These diagonal braces form panels 7 ft. in height and 6 ft. wide, the braces being fastened to the posts by one $\frac{1}{2}$-in. bolt at each end. The 11-ft. posts are placed so as to break joint in the middle of every other panel. The pieces were all cut to size, and holes were bored for the bolts before the timbers were placed in the tower. Two guides, made of 1x4-in. Georgia pine, are placed on the inside of the tower, between which the bucket is made to run. The gin-pole used in erecting the tower was constructed to slide between these bucket guides, so that, as each section of the tower was completed the gin pole could be easily pulled up into the completed section and placed in position for hoisting the timbers for the next section above.

The tower is guyed by four sets of lines; one at the top, and the others spaced further down on the tower. These guys are $\frac{5}{8}$-in. crucible steel cables, and are fastened to "dead-men" buried in the ground. Over the top of the tower the main cable is supported by two 14-in. sheaves, and from this cable the main line of chutes is hung.

The concrete, which is dropped into the hopper at the top of the tower, is released by hand into the Insley chutes, which consist of steel plates bent into a 10-in., three-quarter-round trough. They are hung from the main $1\frac{1}{4}$-in. cableway by blocks and falls. The chutes are built in 10 and 20-ft. lengths, enough sections being provided to provide a maximum length of 450 ft. along the viaduct. At intervals along this line of chutes, line-gates or hoppers are provided, so that this line can be tapped at any convenient point. The main line of chutes is directly over the east face of the conduits; it is therefore necessary when concreting along the westerly line of the conduits to use laterals, connected to the line gates of the main chutes, the lower ends of these laterals being suspended from an auxiliary cableway about in line with the center of the viaduct. It takes from 40 to 60 seconds for the bucket to make a round trip from the mixer to the top of the tower and down again. The maximum rate of mixing and placing concrete is 160 cu. yds. in a ten-hour day, but there has never been occasion to press the plant to a greater capacity. The concrete comes out thoroughly mixed, and the system as a whole has proved very satisfactory.

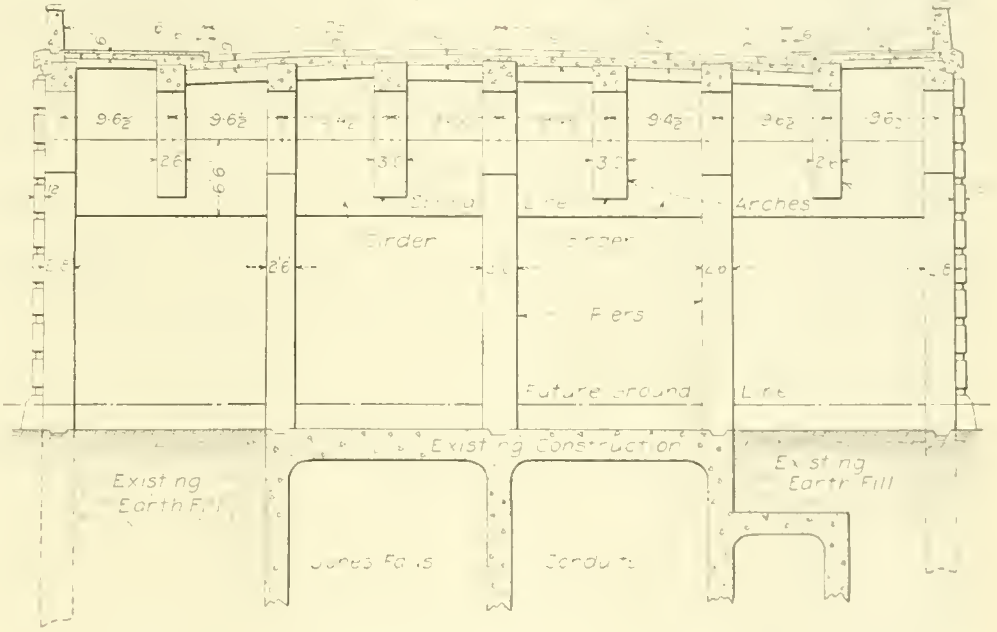


Fig. 1. Cross-Section of Fallsway Viaduct, Baltimore, Md., Showing Type of Construction and Existing Conduits.

The portion of the viaduct north of Chase St. is inaccessible from chutes which are suspended from this tall tower. It was originally planned to build a second tower and plant at Biddle St., for placing the concrete north of this point, but owing to the congested conditions there it was impracticable to build a new plant for this portion of the work. A new tower, 185 ft. high, guyed with three sets of wire ropes, was erected at the north side of the Chase St. Bridge. The concrete is chuted from the top of the old 205-ft. tower into a hopper at the base of the new tower. From this level it is lifted in a bucket up to this second tower, and is delivered to the parts of the viaduct not reached from the first tower.

For the retaining walls for the belt course, and for the parapets at the south end of the viaduct, a small traveling mixing plant is used, this plant being 14 ft. wide by 20 ft. long. A mixer, a hoisting engine, and a 30-ft. tower are mounted on the floor of this car, the concrete being dumped into a bucket, as in the main tower. From the hopper, which is at the top of the tower, the concrete is run through a chute into the forms. This car is never more than 25 ft. from the point where it is desired to concrete, so that one section of chute is sufficient. The materials are supplied by wheelbarrows, the cement being stored in a supplementary cement house near the lower end. The sand and gravel are dumped by the motor truck immediately alongside the traveling mixer.

Another small traveling mixer is used to pour some of the small parapets and sidewalks. It is mounted on a truck, which is built high enough—about 3 or 4 ft.—to enable the concrete to run readily into the forms. This mixer is driven by a gasoline engine, the materials being thrown into it by hand.

A small locomotive crane is used to place the centers for the arches and to move them from place to place.

An electric-driven air compressor is installed under the Eager St. Bridge, and a pipe line is run the entire length of the work for the pneumatic boring machine which is used for the carpenter work.

Along the west side of the walk, just below Eager St., are located the carpenter and blacksmith shops, with the field office and storeroom between them. The carpenter shop is fitted with electric-driven band saws, rip saws, joiners, etc. A blacksmith shop is also provided, where repairs are made and where the steel forms for the coping blocks are constructed. The contractors maintain an office, where the drawings for the forms are made.

CONCRETING AND FINISHING.

The general order of procedure is to pour the piers up to the level of the bottom of the girder, then to pour the remainder of the piers with the girder and the intermediate skewbacks. The arches are then poured, followed by the roadway floor slab, which is poured as soon as the next set of arches ahead is in place. The belt course and the sidewalk slabs are then placed, and finally the parapet and coping blocks are poured.

In the piers a little difficulty was experienced, at first, in preventing the grout from remaining near the surface, causing streaks to show when the piers were hammer dressed. This was overcome by placing wire netting about 1 ft. from the surface, the concrete between it and surface being then placed by hand. The hammer dressing is done with pneumatic hammers, the tools used consisting of sets of four knife edges bolted together, which produce a slightly lined effect. At the beginning, trouble was encountered in hammer dressing the many edges of the piers and the belt courses. These edges had a tendency to break off. After inspecting a great deal of work the engineers decided to rub all edges and finish them sharp, making a line about 1 in. from the edge. The surface was then hammer-dressed up to this line and the edge left sharp, giving a very pleasing and finished effect.

FORMS

The forms for the piers are built of 2-in. lagging, with 2x6-in. vertical studding spaced 30 ins. on centers and with horizontal wales of the same size. Rods are run through the pier, and are held by Universal clamps to keep the wales from spreading. These clamps are also used in the spandrel walls, girders, and wherever there are large surfaces in the form work.

The girder forms are made of the same sizes of lumber as the piers.

The arch centering consists of two trusses for each arch, each truss being made up of two similar halves, bolted together at the center of the arch. There are six panels in each half, averaging 4 ft. in length and from 3 to 4 ft. in depth. The diagonals are made of 2x6-in. pieces and the lower chord of 2x8-in. timbers. The upper segment boards are cut from 2x10-in. lumber. These are joined together by ⅞-in. bolts, with "Ogee" washers, making in all a truss with a large factor of safety. Two-inch lagging is used over these trusses, to form the bottom surface of the arch.

After the arches are poured, the centering is left in place for 28 days and is used to support the joists for the forms for the floor beams and slabs. These forms are made of 1-in. lagging, supported by 2x6-in. joists. The forms for the belt course are held by braces which rest on the centering of the outside arch. All edges of floorbeams, girders and arches are beveled, the amount of bevel varying with the size of the member. The forms for the coping blocks are made of ⅛-in. steel plates, being constructed in 7-ft. sections. The edges are strengthened with small angles, and the entire form is mounted on timber yokes.

PERSONNEL.

The viaduct is being constructed by Claiborne, Johnson & Co., under the supervision of Mr. Calvin W. Hendrick, consulting engineer for the city.

Sewerage

Method and Cost of Constructing an 18 In. Inverted Syphon for Sewer Crossing of Letort Spring, Carlisle, Pa.

Contributed by C. A. Bryan, Resident Engineer.

The construction of the outfall sewer serving the northern and the western section of the Borough of Carlisle, Pa., necessitated making a crossing of the Letort Spring, a stream flowing through the eastern section of the borough, in order to connect this sewer with the main outfall sewer. Figure 1 gaves a sketch plan and profile of this crossing as submitted to the contractors at the time that bids were received for the construction of the lateral sewer system at Carlisle, and shows that it was the original intention to cross the stream by carrying the pipe on the same gradient as that on which the rest of the 18-in. line was laid.

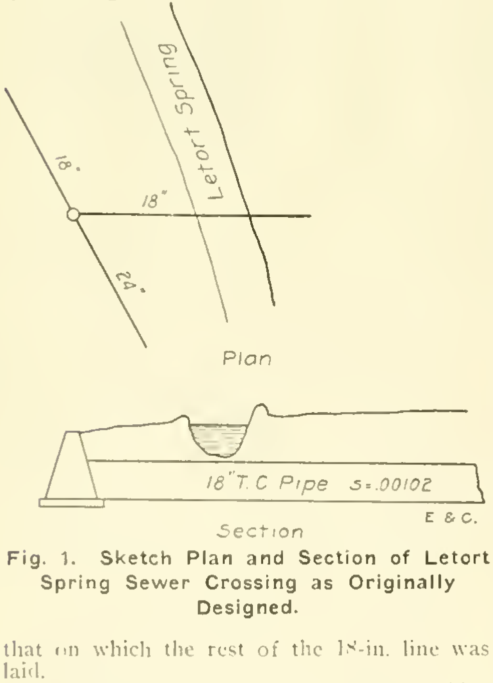


Fig. 1. Sketch Plan and Section of Letort Spring Sewer Crossing as Originally Designed.

The eastern section of the borough, which is drained by the Letort Spring, is quite low and is subject to severe floods. For some time past this subject has been discussed by the borough authorities, and money was finally appropriated to relieve it. A careful study of the conditions proved that relief could be obtained only by lowering the stream and converting the present winding channel into one having a properly graded bottom and properly sloped sides, and a width at the water surface of 40 ft. To permit the execution of these changes at a future date without interfering with the sewer main it was decided to construct an inverted syphon at the place where this sewer crossed the spring. A cross-section of this crossing is shown in Fig. 2.

The width of the spring at the point where the crossing was made was 33 ft. at the water surface. The channel itself was, however, about 15 ft. wide, the remainder of the cross-section being more or less choked with mud and silt, and through this part of the section the velocity of flow was low. The average depth of water in the cross-section was 1.05 ft.

Soundings taken at the crossing showed that the bed of the spring was composed of a stratum of soft mud and clay about 4 ft. deep and underlaying this a stratum of stiff clay through which the soundings were not carried. It was therefore decided to construct this crossing in two sections, and to completely finish and fill in the first section before beginning work on the other.

For convenience a rough foot bridge was first built across the spring at this point. Work was then commenced on the eastern half of the crossing, and a cofferdam built extending from the eastern bank of the spring to a point a little beyond its center. The cofferdam was made about 20 ft. long and 5 ft. wide inside dimensions, and was built by driving two rows of 2-in. plank around the three sides exposed to the water. The plank used were 8 ft. long and were braced by one horizontal waling strip. A wooden maul was used to drive this sheeting and no trouble was experienced in driving it to a sufficient depth. The space between the two rows of sheeting was then filled with a well-puddled stiff clay excavated from the bottom of the stream. Water was then pumped out of the cofferdam by gasoline pumps, while bags filled with sand were piled around the outside of the cofferdam to prevent excessive infiltration. The leaks in this cofferdam were plugged without much difficulty. The trench for the pipe was then excavated and it was found necessary to use tight sheeting throughout its length. The trench was dug about 3 ft. wide inside to inside of sheeting in order to make a proper allowance for clearance of the bells of the pipe. The trench sheeting was held by one line of horizontal waling strips set just below the original bed of the stream. The material excavated from the trench was disposed of on the banks of the spring. When the excavation had been carried to the elevation called for on the plan, it was found that the foundation was not solid and it was therefore carried about 8 ins. deeper to a more solid stratum. The contract called for surrounding the pipe with 6 ins. of concrete and it was decided to further reinforce this by using 1-in. round iron rods spaced 9 ins. apart on centers underneath the pipe. Provision was also made for providing for a sufficient lap for these rods at the end of the section.

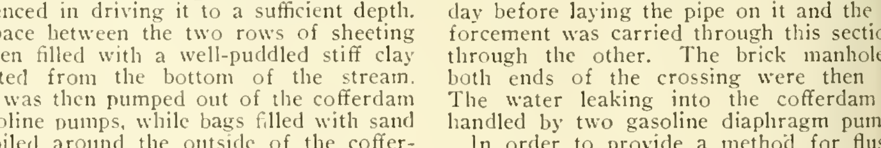


Fig. 2. Cross Section of 18-in. Clay Pipe Inverted Sewer Syphon Under Letort Spring Carlisle, Pa.

The work of excavation was finished late one afternoon and the 8-in. concrete base was placed that same day. This was allowed to set until noon of the next day when it was tested and found to have acquired considerable set so that the 18-in. pipe was laid on it and surrounded with concrete as specified. A tight wooden bulkhead was set in the end of the last pipe laid. Pumping was continued during all of the time of pipe laying in order to insure the inspector that the joints were completely filled with mortar. This was a precaution considered as essential at this crossing as at points where the pipe was laid in dry excavation and without the concrete surrounding it. As soon as these joints had set, however, and the placing of the concrete around the pipe commenced the pumping was stopped. The concrete placed around the pipe was thus placed in water but no flow of the water was observable, and no discoloration of the spring was noted below the cofferdam.

Work on the western half of the crossing was started a few days after the completion of the work just described. This work was handled in the same manner. Some difficulty was experienced in keeping the water out of this cofferdam and considerable time was lost in attempting to make the excavation watertight. As on the other section, the concrete foundation was allowed to set for about a day before laying the pipe on it and the reinforcement was carried through this section as through the other. The brick manholes at both ends of the crossing were then built. The water leaking into the cofferdam was handled by two gasoline diaphragm pumps.

In order to provide a method for flushing out this syphon it was decided to connect the manhole on the west bank of the spring with the spring by a 12-in. cast iron pipe resting on the bed of the latter. This pipe was provided with a 12-in. gate valve which was set in a valve chamber built into the manhole thus providing a method of controlling the quantity of water admitted into the sewer. The other end of this line was provided with a suitable screen.

Work on the construction of this crossing was started on Sept. 12, 1913, and completed on October 1. The itemized cost of this work to the contractor was as follows:

COST OF LABOR.

Item.	Amount
18 hrs. supervision at $0.80	$ 14.4
188 hrs. foreman at $0.30	56.4
40 hrs. carpenters building cofferdam at $0.35	14.0
40 hrs. carpenter helper on above at $0.17½	7.0
280 hrs. excavation at $0.17½	49.0
80 hrs. stopping leaks at $0.17½	14
20 hrs. stopping leaks at $0.20	4.5
153 hrs. driving sheeting at $0.17½	26.7
35 hrs. pipe laying at $0.17½	6.1
11 hrs. pipe laying at $0.27½	3.0
80 hrs. mixing and placing concrete at $0.17½	14.0
5 hrs. mixing and placing concrete at $0.20	1.0
10 hrs. backfilling at $0.17½	1.7
37 hrs. moving pumps at $0.17½	6.4
36 hrs. engineer at $0.40	14.4
44 hrs. mason at $0.30	13.2
41 hrs. mason helper at $0.17½	7.1

25 hrs. cleaning up at $0.17½	4.38
1 hr. cart at $0.27½	0.28
8 hrs. team at $0 45	3.60
3 hrs. hauling 12-in. pipe at $0.45	1.35
Total cost of labor	$268.63

COST OF MATERIALS.

180 ft. B. M. lumber at $25.00	$ 29.50
200 lin. ft. ⅝-in. lumber	4.73
45 empty sacks used in cofferdam at $0.10	4.50
9 cu. yds. of stone at $1.25	11.25
7.5 cu. yds. of sand at $1.75	13.10
68 bags of cement at $0.40	27.20
500 brick at $8.75	21.88
55 gals. of gasoline at $0.17½	9.63
52 lin. ft. 18-in. terra cotta pipe at $0.60	31.20
448 tons 12-in. c. i. pipe at $24.90	12.28
2 manhole frames and covers at $6.70	13.40
10 manhole steps at $0.35	3.50
1 cut stone at $2.25	2.25
1 12-in. valve at $39.50	39.50
Leading pipe into valve	2.50
Steel reinforcement	3.40
4 diaphragms for pumps at $3.25	13.00
Oil for pumps	1.50
25 days of pumping for gas pumps at $1.00	25.00
Total cost of materials	$269.32
Total cost of labor	268.63
Total cost of crossing	$537.95

The itemized cost of constructing the 12-in. ushing main and valve in the manhole on ie western bank of the spring was as follows:

1 hrs. of laborers building cofferdam at $0.17½	$10.68
7 hrs. of mason at $0.30	5.10
3 hrs. of teams hauling supplies at $0.27½	0.83
1 12-in. gate valve at $39.50	39.50
6 brick at $8.75 (M)	3.91
1 cut stone	2.25
2 cu. yds. of stone (crushed) at $1.25	2.50
2 cu. yds. of sand at $1.75	3.50
0 bags of cement at $0.40	4.00
0 ft. B. M. lumber at $26.00	7.80
0 gallons of gasoline at $0.15	1.50
Leading pipe into valve	2.50
Total cost of 12-in. flushing device	$84.10

Deducting the cost of constructing the 12-in. ushing device from the total cost of the ossing gives $453.85 as the cost to the con-actor of constructing this inverted syphon gether with the two manholes and other purtenances.

The length of this crossing was 52.5 lin. ., or the cost of construction amounted to .65 per lineal feet.

The contractor requested extra compensa-on for this work, claiming that the construc-on of the inverted syphon embraced work a nature different from that covered by e plans and specifications and upon which s original bid was premised. The specifica-ns covering this point read as follows:

> Any doubt as to the meaning of this agree-ent, or the annexed specification, or any ob-urity of the plans, will be explained by the ief Engineer. He shall have the option of aking changes in the plans relating to the sition, dimensions or materials, either be-re construction or after it has begun. This cludes the changing of the depths of the wers or appurtenances from that shown upon e plans. The Chief Engineer shall have the al decision on all matters of dispute involv-g the character and amount of work to be ld for under this agreement, and he shall termine all questions relating to the work materials and the construction thereof, and he shall, in all cases, decide every question which may arise relative to the execution of this contract on the part of the contractor, and his estimate and decision shall be final and conclusive, and such estimate and decision, in case any question shall arise, shall be a condition precedent to the right of the contractor to receive any money under this agreement.

The decision duly reached by the engineer was: First, that the contractor in his contract with the borough agreed to carry this 18-in. sewer across the Letort Spring in accordance with the plans shown and to surround it with concrete. Second, that the substitution of the inverted syphon for the crossing at grade as shown on the plan did not require the contractor to purchase any more material to complete this crossing than would have been required had the crossing been constructed as shown upon the plans. Third, that the extra expense to which the contractor was put was merely a question of the extra cost of labor necessary to lay the pipe at the increased depth. Finally, that in as much as the contractor would have been put to some expense to make the crossing as shown on the plans, it would seem fair and equitable to him if the borough should pay one-half of the extra cost of the labor necessary to complete the work at the increased depth. The final estimate rendered the contractor therefore contained an item of $133.72 as representing one-half the increased cost of labor required to construct this inverted syphon. The cost of constructing the 12-in. flushing device was assumed in full by the borough and 15 per cent added thereto as the contractor's profit.

The work described was a portion of that embraced in the construction of the lateral sewer system at Carlisle, Pa. This work was designed by and constructed under the supervision of Mr. T. Chalkley Hatton, Chief Engineer of the Milwaukee Sewerage Commission, Milwaukee, Wis. The contractor for this work was The H. C. Brooks Company, Inc., Martinsburg, W. Va., and the work was carried on under the supervision of Mr. H. M. Sherwood. The writer was the resident engineer in charge of construction.

The Fertilizing Value of Sewage and Sewage Sludge.

From time to time attention is called by various people, scientific and otherwise, to the great value of the fertilizing materials in sewage and the waste of these materials by all modern methods of sanitation and sewage disposal. These methods will undoubtedly continue to prevail, and the fertilizing materials of sewage will continue to be wasted, until some economical method of reclaiming them is devised. In other words, the prevention of this waste concerns essentially the value of these materials in a given volume of sewage when compared with the cost of extraction. Enough material of a fertilizing or other value must be extracted by any process to more than pay for the cost of the process if it is to be employed. This problem will be solved eventually, but the difficulties are great, and only slight progress has thus far been made in its solution. The subject was rather fully discussed by Mr. H. W. Clark, Chemist of the Massachusetts State Board of Health, in the monthly bulletin of the Board for December, 1913. Numerous data bearing on the subject were presented in the article. We reprint Mr. Clark's conclusions as follows:

CONCLUSIONS.

(1) The total amount of fertilizing and fatty matters in each 1,000 gals. of representative American sewage is not worth above 6 or 8 cts. Of this, about half is represented by the ammonia in solution, and there is no method by which this material can be utilized except by application of the sewage to land. All experience, covering many years with hundreds of well-operated sewage farms ranging in size from a few acres to the vast 39,000-acre farm at Berlin, Germany, has shown that only under the most favorable conditions can the return from these farms be made to pay operating expenses, and an instance is yet to be cited where these returns pay both the cost of operation and interest on the capital invested. The exceptions, perhaps, to this are certain tracts or farms in regions of low rainfall and where the sewage is valuable as a liquid; that is, for real irrigating purposes. The Wolverhampton farm is undoubtedly a representative English farm as regards good management and cost of operation, and at this farm the actual total cost of sewage treatment is $49 per 1,000,000 gals.

(2) Much of the valuable fertilizing and fatty constituents in sewage is found in the matters in suspension. Average American sewage contains, perhaps, about 2,400 lbs. of sedimentable matter in 1,000,000 gals., and the nitrogen, fatty matters, etc., in this 2,400 lbs. of sludge are worth, approximately, $15 or $18. In order, however, to reclaim this valuable material, the sludge must be dried, pressed and also subjected to a process for the separation of grease from the fertilizing constituents in the remaining body of the sludge. Only by this separation can the grease become an article of commerce and the sludge of real agricultural value. This fact is well established by long experience and many investigations. When the fatty matters are separated by any known process, this procedure is costly. Only in a few places as yet is such separation attempted as a commercial enterprise, and the profitableness of the works at these places is as yet doubtful. When the sludge is freed, or practically freed, from fatty matters it consists of a large weight of inert mineral and organic matter mixed with comparatively small weight of fertilizing matter, hence the cost of carriage is great, even when it is well dried. It has also been well proved that the nitrogen, phosphoric acid, etc., present are generally in a less assimilable form than the same bodies in ordinary commercial fertilizers. The sludge has value, however, and as the processes of drying, pressing and fat separation are improved, and also as nitrogen advances in price, as seems inevitable, sewage sludge will become of greater agricultural value than it is at present, especially as the basis of a fertilizer enriched by the addition of potash, phosphates, etc.

General

upreme Court Decision on the King's County Lighting Co. Rate Case, New York.

Appeal by permission from an order of the pellate Division of the Supreme Court in e First Judicial Department, made in a cer-rari proceeding to review the determination the Public Service Commission for the rst District of New York in fixing the max-um rate for gas to be charged by the Re-or in the 30th Ward of the Borough of ooklyn.

The relator was organized in 1904 and rged with the Kings County Gas & Illuminating Co., which was organized in 1889 and began supplying gas in October, 1891.

The Appellate Division held that the determination of the Commission was wrong in three respects, which are presented by four questions certified for review, viz:—

(1) Was the relator entitled, upon the facts shown in the record, to have the Commission make an allowance for going value in determining the value of the relator's property used in the public service?

(2) Was the relator entitled, upon the facts shown in the record, to have the cost of reproduction of paving now in the streets but not in place at the time the mains were laid, allowed for in ascertaining the value of its property used in the public service?

(3) Was the relator entitled, upon the facts shown in the record, to have the cost of reproduction of paving now in the streets but not in place at the time the mains were laid, allowed for in ascertaining the capital actually expended?

(4) Was the Commission entitled, upon the facts shown in the record, in ascertaining the amount which should constitute a proper return, to consider as part of what accrues to the relator as gross receipts of its yearly operations the annual appreciation in the value of its land?

George S. Coleman & Oliver Semple for Appellants.
O'Brien, Boardman & Platt (Morgan J. O'Brien, James F. Meagher, Swinburne Hale, of Counsel), for Respondent
Miller, J.:

It is now generally recognized that "going value" as distinct from "good will" is to be considered in valuing the property of a public service corporation either for the purpose of condemnation or rate making, but there is a wide divergence of view as to how it is to be considered. The Commission in this case says it was taken into account in valuing the plant as a "going" and not as a "defunct or statio" concern and that it was also considered in fixing the fair rate of return: The Appellate Division says that there is no proof of the latter fact in the record. Thus the first question certified requires us to decide whether "going value" is to be appraised as a distinct item, or whether it is sufficient to regard it as something vague and indefinable to be given some consideration but not enough to be estimated. The valuation of the physical property was determined by ascertaining the cost of reproduction less accrued depreciation. Preliminary and development expenses prior to operation were included, but no allowance was made for the cost of developing the business. By that method, the plant was valued in a sense as a "going concern," in other words, "scrap" values were not taken; but to say that that sufficiently allows for "going value" is the same as to say that "going value" is not to be taken into account. The problem is to determine what is fair to the public and the company. The public is entitled to be served at reasonable rates, and the company is entitled to a fair return on its investment, on the value of the property used by it in the public service. (Smith v. Ames, 169 U. S. 466; San Diego Land Co. v. National City, 174 U. S. 739; San Diego Land & Town Co. v. Jasper, 189 U. S. 439; Minnesota Rate Cases, 230 U. S. 252.) It would have been entitled to a return on the valuation adopted by the Commission, if it had no customers, but was just ready to begin business, whereas it had a plant in operation with an established business, which every one knows takes time, labor and money to build up.

If going value is capable of ascertainment, it will not do for the Commission vaguely to consider it in fixing the fair rate of return. That no appreciable allowance was made in this case is shown both by the rate fixed and by the following statement in the opinion of the Commission:

> It should be noted that the plant has been in operation for nearly twenty years, and it might be argued with considerable force that two decades should be sufficient for the company to recoup its early deficiencies below a fair rate of return, if any such deficiencies ever existed. If the company has not recouped itself by this time, under such circumstances it is doubtful whether the present consumers ought to be burdened for this reason.

The first question certified then resolves itself into two heads: (1) Is "going value" a distinct item to be appraised and included in the base upon which the fair return is computed? (2) Was the evidence in this case sufficient to justify an allowance for it?

The opinion of Mr. Justice Clarke below saves me the necessity of citing and analyzing the cases bearing on the first branch of the question. We concur fully in what he has said on the subject and in his conclusion that there is no "logical difference between allowing 'going value' in the valuation of a plant when it is to be taken entirely by the public and allowing the same element when valuing the same plant for rate making purposes." A case since decided should be added to the rate cases cited by him in which 'going value" has been allowed; i e., Public Service Gas Co. Board of Commissioners, 87 Atl. Rep. (N. J 651).

It is no answer to say that in condemnation cases the exchange value is taken, and that that depends on the rates charged, the thing to be determined in rate cases. Of course, a rule of valuation might be adopted in a condemnation case, which would not work in a rate case; but if the cost of reproduction less depreciation rule be adopted as appears to have been done in National Waterworks Company v. Kansas City (62 Fed. Rep. 853), and Omaha v. Omaha Water Company (218 U. S. 180), the leading condemnation cases in the Federal Courts in which "going value" was considered, it is impossible to see why the "going value" could not be determined in both classes of cases in precisely the same way.

The difficulty of determining the "going value" will not justify the disregarding of it. Rate making is difficult. But that will not justify confiscation. The difficulty, however, will lessen, as it does in most cases, when we cease to think about the subject vaguely. What then is going value, and how is it to be appraised?

It takes time to put a new enterprise of any magnitude on its feet, after the construction work has been finished. Mistakes of construction have to be corrected. Substitutions have to be made. Economies have to be studied. Experiments have to be made, which sometimes turn out to be useless. An organization has to be perfected. Business has to be solicited and advertised for. In the case of a gas company, gratuitous work has to be done, such as selling appliances at less than a fair profit and demonstrating new devices to induce consumption of gas and to educate the public up to the maximum point of consumption. None of those things is reflected in the value of the physical property, unless, of course, exchange value be taken, which is not admissable in a rate case. The company starts out with the "bare bones" of the plant, to borrow Mr. Justice Lurton's phrase in the Omaha Waterworks case, *supra.* By the expenditures of time, labor and money, it coordinates those bones into an efficient working organism and acquires a paying business. The proper and reasonable cost of doing that, whether included in operating expenses or not, is as much a part of the investment of the company as the cost of the physical property.

The investors in a new enterprise have to be satisfied as a rule with meager or no returns, while the business is being built up. In a business subject only to the natural laws of trade, they expect to make up for the early lean years by large profits later. In a business, classified among public callings, the rate making power must allow for the losses during the lean years, or their rate will be confiscatory and of course will drive investors from the field. In the former class, the value of the established business is a part of the "good will" and may be determined by taking a given number of years' purchase of the profits or exchange value may be considered. In the latter class, a different rule must be adopted.

Referring again to the Ames case, *supra*, the public is entitled to be served at reasonable rates and the corporation is entitled to a fair return on the property used by it in the public service, no more, no less, always assuming, of course, that the return is computed on a proper valuation. That was not made so by statute, but was the rule at common law, which justifies legislatures and commissions in fixing rates. If then a public service corporation has received more or less than a fair return, it has received more or less, as the case may be, than was its due, irrespective of whether a rate had been fixed by public authority. If a deficiency in the fair return in the early years was due to losses or expenditures which were reasonably necessary and proper in developing efficiency and economy of operation and in establishing a business, it should be made up by the returns in later years. If there was a fair return from the start, the corporation has received all it was entitled to, irrespective of how much of the earnings may have been diverted to the building up of the business.

To view the matter in another aspect, take the case of a public service corporation with a plant constructed just ready to serve the public. It is going to take time and cost money to develop the highest efficiency of the plant and to establish the business. Three courses seem to be open with respect to rate making, viz: (1) To charge rates from the start sufficient to make a fair return to the investor and to pay the development expenses from earnings, a course likely to result in prohibitive rates except under rare and favorable circumstances; (2) to treat the development expenses as a loss to be recouped out of earnings but to be spread over a number of years, in other words as a debt to be amortized—that involves complications, but would seem to be fairer to the public and certainly more practical than the first; (3) to treat the development expenses, whether paid from earnings or not, as a part of the capital account for the purpose of fixing the charge to the public. The last course would seem to be fairest to both the public and the company, as well as the most practical.

It may be, as is urged, that a well conducted enterprise will charge the cost of developing the business to operating expenses, and that it would open the door to an over issue of securities to permit the capitalization of early losses. In answer, it is sufficient to say that we are dealing, not with proper methods of bookkeeping, not with proper capitalization upon which to issue securities, but solely with the fair return which the company is entitled to receive from the public. Treating a reasonably necessary and proper outlay in building up a business as an investment for the purpose of determining the fair rate of return to be charged is far from holding that it should be treated as capital, against which securities might be issued.

We do not say as matter of law that the third course above outlined should be adopted as an original proposition. That may present a question of economics, depending on the particular conditions involved. The Commission in this case had to determine the rate to be charged, not by a new company with no business, but by an old company with an established business. The first question, therefore, to determine on this branch of the case was whether the company had already received a fair return as its investment. If it had received such return from the start, or if in later years it had received more than a fair return, the public would already have borne the expense of establishing the business in whole or in part and to that extent the question of "going value" for the purpose of fixing a present rate would be eliminated. For it must be constantly kept in mind in dealing with this problem that the company is entitled to a fair return and no more. If it has already had it, that is the end of the matter. If it did not receive a fair return in the early years, owing to the establishment of the business, a subsequent rate must allow for that loss or it will be confiscatory. Now, no dividends appear to have been paid by the original company, or by the relator prior to 1907. Assuming a reasonable need of the service from the start and that the failure to pay dividends was not due to bad management, an accumulation of a surplus or undivided profits, the investment of earning in permanent additions or betterments allowed for in the structural valuation, or to other causes besides those under consideration, none of which is assured, it would seem plain that "going value" was an element in this case which the commission was required to determine in making an appraisement on which to compute the fair return to which the company is entitled.

It is urged that an unprofitable business will thus have a greater value for rate purposes than one profitable from the start. That again overlooks the fundamental consideration that a public service corporation is entitled to a fair rate of return from the beginning of its investment and no more. If the shareholders have been deprived of a fair return on their investment because of the time and expense reasonably and properly required to build up the business, they have, to the extent of that deprivation, added to their original investment and are entitled to a return upon it. If, however, a fair return in addition to the expense of building up the business has been earned from the start, the public, not the shareholders, have paid the development expenses. We are dealing not with exchange values but with the value upon which the company is entitled to earn a return. In this connection, it is to be observed that the statements in the opinions of the courts, in reference to computing the fair rate of return on present values, have for the most part been made in cases in which the precise question under consideration was not directly involved and in which no attempt was made to limit the elements composing the problem. Manifestly, a rate computed on the cost today of reproducing the bare plant would not be fair. Experience is proverbially expensive. With the advantage of that experience the same or an equally efficient plant could be constructed today at a cost much below the actual and necessary investment of the company in both plant and experience. Indeed, wholly apart from the intangible thing called the going business, the reproductive value today of the physical property would not necessarily include the actual and legitimate investment in tangible property which may have been entirely replaced, not because of depreciation, but to meet advances in mechanical science, new conditions and increasing demands not reasonably to have been foreseen at the start. I am not now speaking of replacements made with fresh capital, about which there is no question in this case. The term "going value" though not exactly defined, has been used quite generally to comprise the elements not included in the structural value of the property in its present condition. The term is not important. The point is that in some manner and under some appropriate heading a due allowance must be made for the investment in those elements. No inflexible rule will in the long run be just both to the public and the corporation. The right to limit the corporation to a fair return fixed by public authority necessarily involves the correlative right in the corporation to be assured of that fair return during all the time that its capital is employed in the public service. The statute governing this case (Section 72, Public Service Commission Laws), provides:

> In determining the price to be charged for gas * * * the commission may consider all the facts which in its judgment have any bearing upon a proper determination of the question although not set forth in the complaint and not within the allegations contained therein, with due regard among other things to a reasonable average return upon capital actually expended and to the necessity of making reservations out of income for surplus and contingencies.

Of course a reasonable need for the service from the start and reasonably good management are assured. While, within reasonable limits, service may be provided for anticipated needs, a company should not construct a plant in a wilderness and, after a city has been built around it, expect to recoup its losses while waiting, nor should it expect to recoup losses from bad management. I do not include in the latter mere mistakes or errors in judgment which are almost inevitable in the early stages of any business. The fair return is to be computed on the actual investment, not on an overissue of securities, and the failure to pay dividends to the investors must be due to the causes under consideration, not to an accumulation of a surplus or to expenditures for permanent additions or betterments, which are included in the appraisal of the physical property—in other words, the actual net earnings are to be taken. Making proper allowance for the matters just considered and perhaps for others which do not occur to me, I define "going value" for rate purposes as involved in this case to be the amount equal to the deficiency of net earnings below a fair return on the actual investment due solely to the time and expenditures reasonably necessary and proper to the development of the business and property to its present stage, and not comprised in the valuation of the physical property.

It may be conceded that going value has no existence apart from tangible property and that commercially there is but one value, that of the property as a whole, but as the rate can not be made to depend upon the exchange value, which would in turn depend upon the rate, it would seem to be necessary to appraise the physical property and the going value separately, and of course that is the case if the cost of reproduction rule be adopted.

It remains to consider how going value is to be appraised. That presents a question of fact, the determination of which is primarily within the province of the rate making body. It is proper for this court, however, to indicate a permissible method or methods, as in People ex rel. Jamaica W. S. Co. v. Tax. Com. (196 N. Y. 39), it indicated a rule for valuing a special franchise for taxing purposes, which has generally been followed.

Obviously, the most satisfactory method is to show the actual experience of the company, the original investment, its earnings from the start, the time actually required and expenses incurred in building up the business, all expenditures not reflected by the present condition of the physical property, the extent to which bad management or other causes prevented or depleted earnings, and any other facts bearing on the question, keeping in mind that the ultimate fact to be determined is not the amount of the expenditures, but the deficiency in the fair return to the investors due to the causes under consideration. The business in this case was twenty years old, the books of the old company were not available, and it is of course problematical whether, if produced, they would have shown the necessary facts. The question, therefore, had to be determined, as all questions of fact have to be, by the best evidence available. Here, I may repeat that mere difficulty in the proof would not justify a confiscatory rate. The value of the physical property was shown by opinion evidence as to the cost of reproduction. The same kind of evidence was given by two witnesses for the relator as to the cost of building up the business to its present state. In the appellant's brief, it is said that they were men "of mature age and much experience." One witness said that $30 a meter was an arbitrary sum usually adopted by engineers as the cost of building up such a business, and he put the cost in this case at $600,000. Another witness estimated the "going value" to be $781,916. He explained at length how he arrived at that figure. He assumed the existence of two plants, situated exactly alike in the same community and with the same physical property: (1) The actual plant with its established business; (2) a suppositional plant with no business. He then estimated the length of time and expense required by the second plant to develop its business to the stage and revenue of the actual plant. There would appear to be as good ground for admitting the opinion of a qualified expert on such a subject as on the cost to reproduce the physical property. Of course, the Commission was not bound by that evidence. It had in addition the experience of the relator and its predecessor as to payment of dividends, the amount of capitalization of both, and the value of the physical property in its present condition determined as above stated. With nothing opposed to those facts and the opinion evidence, it was not justified in ignoring the element of "going value" or of merely attaching some inappreciable importance to it (See Bonbright v. Geary, 210 Fed Rep 44, 51, 56). The first question certified must be answered in the affirmative

Upon the next two questions we disagree with the learned Appellate Division.

In determining the cost of reproduction, the Commission allowed $12,717 as the cost of restoring the pavement as it existed when the mains and service pipes were laid in the streets The relator claimed an allowance of at least $200,000 for the cost of restoring pavements subsequently laid on the theory that that cost would have to be incurred if the mains were to be laid today. But the new pavements in fact added nothing to the property of the relator. Its mains were as serviceable and intrinsically as valuable before as after the new pavements were laid. The controlling considerations under the preceding point also determine. The rights of the public are not to be ignored. The question has a double aspect. What will be fair to the public as well as to the relator? (Smith v. Ames, *supra.*) Should the public pay more for gas simply because improved pavements have been laid at public expense? It is no answer to say that the new expensive pavements suggest improved conditions, which, though adding to the value of the plant, will not, by reason of the greater consumption, add to the expense per thousand feet of the gas consumed. The public are entitled to the benefit of the improved conditions, if thereby the relator is entitled to supply gas at a less rate. The relator is entitled to a fair return on its investment, not on improvements made at public expense. It is said that the mains will have to be relaid. So will the new pavements, and much oftener. Both might possibly be relaid at the same time. The case is not at all parallel to the so-called unearned increment of land that the company owns. It does not own the pavements, and the laying of them does not add to its investment or increase the cost to it of producing gas. The cost of reproduction less accrued depreciation rule seems to be the one generally employed in rate cases. But it is merely a rule of convenience and must be applied with reason. On the one hand it should not be so applied as to deprive the corporation of a fair return at all times on the reasonable, proper and necessary investment made by it to serve the public and on the other hand it should not be so applied as to give the corporation a return on improvements made at public expense which in no way increase the cost to it of performing that service.

The Appellate Division felt bound by the decision of the United States Circuit Court in the Consolidated Gas case (157 Fed. Rep. 849), and it is true that such an allowance was made in that case. But the United States Supreme Court held in that case (212 U. S. 19) that the rate established was not confiscatory, and did not pass on the propriety of that allowance. What was said in the opinion on the subject of present value was merely a general statement having no necessary relation to the question now under consideration.

We agree with the Appellate Division that annual increase in the value of land is not income. Of course, under the rule of the Ames case (*supra*) land might become so valuable as to require its use for other purposes and as to make a rate based on it unfair to the public. The present is not such a case.

The first question should be answered in the affirmative: the second, third and fourth in the negative, and the order of the Appellate Division, insofar as it remits the proceedings to the Commission, should be affirmed

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Methods of Designing Shafts in Earth for the City Tunnel of the Catskill Aqueduct.

The city tunnel of the new water supply system for New York City has a total length of 17 miles from Yonkers, N. Y., to terminus in Brooklyn. For construction it was divided into four sections and six construction shafts were employed on each section All of the tunnel is in rock. On three of the contracts, the rock in general lay near the surface, and the problem of sinking shafts was the regular one of sinking in the open. But in the fourth contract, known as No. 67, which extends from Union Square under the East River to Brooklyn, all of the shafts were located at points where the surface of the rock lay well below the ground water level, and the problem of the shafts became one of sinking through wet ground The method of designing for these conditions the portion of the shafts above the rock is discussed in a paper by R. N. Riegel in the "Cornell Civil Engineer" for March, 1914, and from this paper we extract the following points:

SELECTION OF TYPE OF STRUCTURE

A number of considerations governed the selection of the type of structure. The principal ones may be briefly outlined as follows:

(1) Of paramount importance was the

requirement that the shafts must permit the construction and safe use of permanent waterways through them to connect with the city mains. These connections must be absolutely safe against all the contingencies which the mind of man could reasonably foresee.

(2) The depth of the surface of the underlying rock was determined by borings to be from 20 to 108 ft. below mean ground water level. The surface elevations were in most cases well above the mean ground water level. The total distance to be passed through before reaching rock therefore varied from about 55 ft. in the case of shaft 19 to about 145 ft. in the case of shaft 23. At shaft 20, discussed in detail later, a depth of 120 ft. was obtained. In every case the borings revealed sand as the material to be penetrated between the rock and the ground water level. This material when saturated, as it is below ground water level, would flow at the slightest opportunity, producing certain trouble.

(3) The tunnel alignment having been placed almost entirely under the city streets in order to avoid endless right of way adjustments, the shafts must of necessity be close to buildings and other structures. These must be disturbed as little as possible by the sinking of the shafts, on account of the damage to life and property which might result.

(4) While the tunnel itself, lying 150 ft. below the surface of the solid rock, may be considered safe from interference due to the construction of other works, the shafts above rock are not so immune. Subways may be built close by, caissons for the foundations of buildings may be sunk in the vicinity, and other subterranean work may be carried on, all with the ever present possibility of accident and sudden readjustment of the soil conditions.

It was concluded that the application of the pneumatic caisson method of construction satisfied all of these considerations most completely. By the employment of a hollow circular caisson of reinforced concrete, a large degree of freedom and security in the arrangement of the waterways could be obtained, and the most economical structure would be obtained capable of withstanding internal and external pressure. Sinking through the depths and material to be encountered was within the limits of pneumatic caisson practice. And this form of construction also permitted an operating and connecting chamber to be superimposed upon the top of the shaft, while its bottom could be securely anchored to the rock. This last feature, together with the ability to reinforce the structure thoroughly between chamber and rock, afforded the desired stability against possible future disturbance. A type of shaft which could be most readily constructed by the employment of a hollow pneumatic caisson of reinforced concrete was therefore decided upon, and the design was worked out upon the hypothesis that the pneumatic caisson method would be pursued in construction.

The diameters of the caissons were determined by different criteria. In the case of Shafts 23 and 21, the quantity of water to be delivered required two risers of 6 ft. clear diameter. The clearances necessary for erecting steel riser interlining inside the shaft led to the adoption of an inner diameter of 18 ft., while the thickness of the shaft casing was taken at 3 ft. after an approximate study of the stresses involved. In the case of shafts 19, 20 and 22, where such large connections were not required, the inside diameter of 15 ft. and 4 ins. was determined by a study of the clearances required for the use of a suitable hoisting gear in the solid rock section below. In these cases, the smaller diameter of shaft justified the use of a smaller thickness of shell, and 2 ft. was therefore adopted as this dimension

LOADS TO BE CARRIED.

Construction Loads.—By construction loads I mean the loads which come upon the structure during its construction and to resist which it must be proportioned, but from which it will be free during the period of operation and use. In determining the proportions of the shafts, the loads of this class were investigated first.

The first load investigated was that due to the "hanging" of the caisson, as it is sometimes called. It has occurred in this class of work that a caisson has been gripped so tightly by the surrounding earth at the top that it was prevented from sinking, and, being deficient in tensile reinforcement, the structure actually parted. Inasmuch as the depth of the rock was definitely known in this case, it was a simple matter to determine the proportion of steel needful at any point in question.

The other construction loads included the air pressure upward upon the roof of the working chamber, and the bursting loads which would arise from filling the interior of the caisson with wet sand or other heavy material to provide a sinking load. The former of these required but little investigation, but a "hoop" reinforcement had to be determined to resist the latter effect.

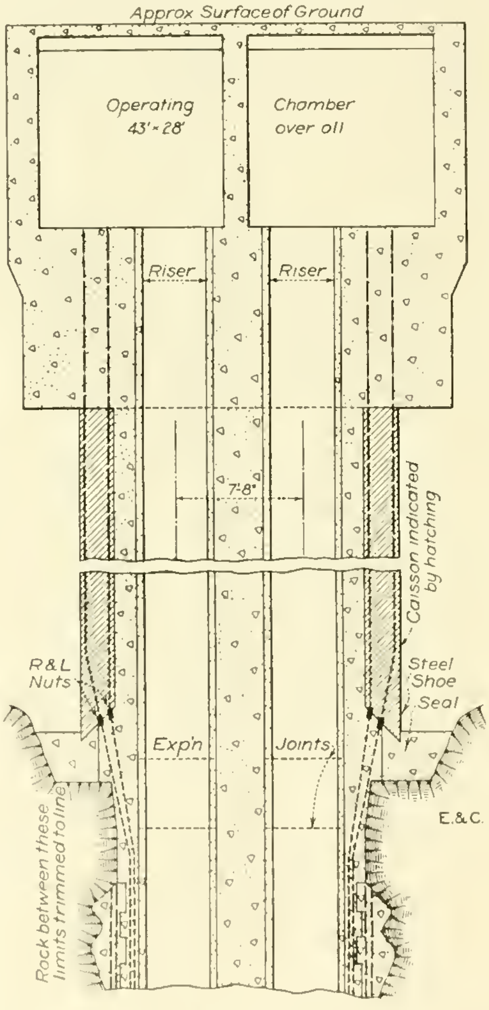


Fig. 1. Reinforced Concrete Shaft Structure for City Tunnel, Catskill Aqueduct.

Operation Loads.—The first of these to be considered is the dead load of the shaft and rectangular operating chamber, with the live load upon the roof of the same. When the latter is applied at the end of the chamber a bending moment tends to be produced in the shaft structure.

Then the tendency of the water in the waterways to burst the shaft was considered. While the water was carried in riveted steel pipe risers of ⅜ in. to ⅝ in. in thickness, the net head of 295 ft. inside the risers was bound to have an effect upon the hoop reinforcement of the shaft.

The principal load in its effect upon the determination of the steel in the caisson, although the least probable in its occurrence, is the unbalanced earth pressure upon the side of the shaft which may arise from the disturbance of the surrounding soil. During the study of the work an accident occurred in the excavation for the foundations of the Municipal building, where some sheeting gave way, causing some settlement in surrounding structures. This suggested the loading which was employed in caring for this feature of the design. It was assumed to be the pressure which would be produced upon the projected area of the chamber and shaft, were all the material upon the opposite side of the structure removed to a depth of 50 ft. from the surface. It was assumed that, below the 50 ft. point, the pressure would be uniform around the periphery of the shaft. This loading produces a constant shear below the 50 ft. point, but the bending moment of course increases with the depth. Heavy reinforcement of vertical rods to resist moment, and of "hoop" steel to resist shear must therefore be provided.

With a tentative scheme of principal reinforcement worked out certain investigation of the relative importance of the above loads was studied.

An investigation was made of the relative effect of the bursting pressure upon the hoop steel and the riser interlining, based upon the comparative distortions of these elements of the structure, with the full static head applied inside the risers. It was found that the riser steel resisted the pressure to a much greater extent than the hoop steel, there being a much greater proportion of the former, per unit of height, than of the latter. With unit stresses in the riser steel of about 12,000 lbs. per square inch, the stress in the hoop steel amounted to only about 2,000 lbs. per square inch.

The feasibility of various types of roof for the working chamber was investigated, and satisfactory types worked out for steel, timber and reinforced concrete construction. The contractors subsequently elected to employ the last named type of roof, cutting it out after the sealing of the caisson had been completed.

Investigation was also made of the quantity of steel needful to resist "hanging" as described above, and to resist the internal pressures due to the sinking weight of sand and water. These quantities were found to be relatively small.

As the result of these investigations it was learned that the reinforcement against unbalanced earth pressure would be so heavy that no other vertical reinforcement would be necessary, and that it would be necessary to reduce the hoop steel stresses for sheer by 2,000 lbs. to allow for the hursting effect of the water in the risers. The safe stress in the vertical steel was taken at 12,000 lbs. per square inch and in the concrete at 500 lbs per square inch above the depth of 50 ft. below the surface. Upon the ground that below this level the probability of heavy stress was very remote, the unit stresses adopted in determining the steel proportion were gradually increased to approximately double the above values at depths of 80 to 90 ft. At these depths the maximum amount of steel which it was regarded practicable to place in the concrete would be reached and computations below this point were considered superfluous. The unit stress in the hoop steel was taken at 10,000 lbs. per square inch. In conformity with the usual practice of the department, the elastic ratio n was given the value of 12.

METHOD OF DESIGN.

The method pursued in determining the dimensions of the shafts may best be shown by following the procedure in detail as it was applied to a typical case. Shaft 20 at Delancey and Eldridge streets, Manhattan, will be taken for the purpose of illustration and the method of design followed in detail.

The load assumed for the design has been given above but is more fully stated as follows: The earth on one side of a vertical plane parallel to the maximum dimension of the operating chamber and passing through the center of the shaft was conceived as being removed. The horizontal pressure of the earth upon the opposite side of this plane was taken at 0.4 of the intensity of vertical pressure, the weight of earth being 100 lbs. per cubic foot. At a depth of 10 ft., therefore the intensity of horizontal pressure was 400 lbs. per square foot and at 40 ft. it was 16,000 lbs. per square foot.

From reference to Fig. 1 it will be seen that the chamber is quite long and that its depth to top of shaft is about 23 ft. The ordinary principles of mechanics lead to a curve of bending moments corresponding to the following tabulation:

Depth in ft.—	Moment in ton-ft.
10	300
20	1,200
30	3,120
40	6,333
50	11,070
60	26,700
70	22,400
80	28,000

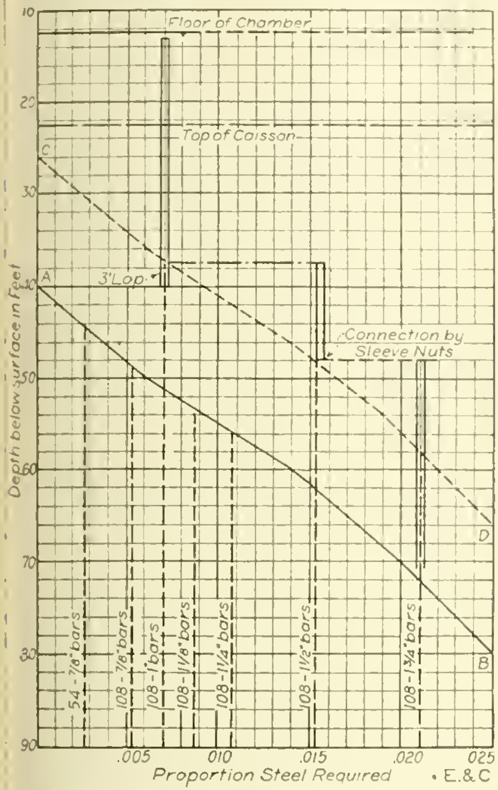


Fig. 2. Diagram for Apportioning Reinforcement in Shaft Structures for City Tunnel, Catskill Aqueduct.

It will be observed that the increment of moment below the depth of 50 ft. is constant; that is, that this portion of the moment curve would be a straight line. This condition arises from the limitation of the removed section to the depth of 50 ft. This loading may appear to be a severe one, but it will be realized, since the earth below the 50 ft. level cannot be regarded as offering a condition of "fixed support" that the scheme of loading will impose severe stresses upon the entire shaft and that the capacity of the structure will thereby be developed to resist irregular unbalanced pressures at any point in its depth. Now, it must be borne in mind that the construction of the shaft involved the sinking of a hollow pneumatic caisson which, after sealing, would remain open during the excavation and lining of the tunnel. When the latter should have been completed, the steel risers would be installed and the space around them filled with concrete. The shrinkage of this concrete and the imperfection of the bond between old and new concrete made it appear extremely improbable that the caisson and its filling should ever act as a unit in the resistance of stress. Moreover, investigation showed that over the greater portion of the shaft the resultant force line would be so eccentric that little difference would arise between calculation of the section in full and the calculation of a hollow cylinder with walls equal to the caisson in the thickness. The analogy between such a structure and the reinforced concrete chimney is obvious. Inasmuch as several methods of analysis of the latter have been developed, these were looked into, and the method worked out in Turneaure and Maurer's "Principles of Reinforced Concrete Construction," 2nd ed., Chap. XI, was adopted as the most feasible one to pursue in this case. Omitting the development of the theory of the above book, we may pass to the following, Table I, in which we have:

D = depth of a horizontal section below surface of ground.

W = weight of structure above section in tons.

M = bending moment at section, in ton-feet.

f'_c = maximum compressive stress in concrete.

f_s = maximum tensile stress in steel.

f_c = concrete stress at point maximum compressive stress in steel.

A = area of section 108.9 sq. ft. = 15,682 sq. in.

e = eccentricity of resultant force = $\frac{M}{W}$

r = mean radius of steel reinforcement.

p = ratio of steel to concrete.

m = factor used in diagram = $\frac{f_cA}{W}$

In Table I the stresses are computed from the diagrams of the text book for the 30 and 40-ft. sections upon the assumption that no steel was present. The steel if present would not be stressed in tension since the ratio $\frac{e}{r}$ does not materially exceed a value of 0.50.

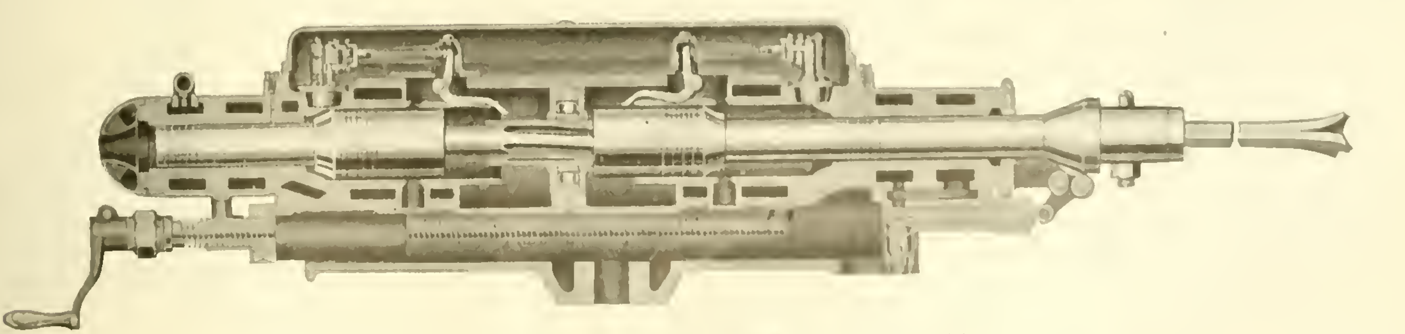


Fig. 1. Sectional View of Gasoline Rock Drill

TABLE I.

D	W	$\frac{W}{A}$	M	$\frac{e}{r}$	m	f_c	f'_c m	p	f_s	e
30	1,199	153	3,120	0.30	1.6	245	264	.000	000	26
40	1,409	179	6,333	0.52	2.04	365	386	.000	0app	4.5
50	1,619	206	11,070	0.79	2.62	540	600	.006	4,800	68
60	1,829	233	16,700	1.05	2.92	680	750	.014	8,800	9.1
70	2,039	259	22,400	1.26	3.13	810	900	.020	12,200	11.0
80	2,249	287	28,000	1.44	3.27	940	1,050	.025	15,000	12.4

Below the level of 40 ft. a different plan was pursued. Preliminary trials having shown that the concrete stresses would govern the design, in that they would increase more rapidly with the depth than would the steel stresses, a schedule of values of f'_c was assumed as indicated in the table. These values led to further assumed values for f_c and m and the values of f_s and p were then obtained from the diagram of Case II. The assumed quantities were then checked back from these results.

The values of p obtained were then plotted upon the diagram of Fig. 2 and curve AB was drawn through them. Ordinates corresponding to various combinations of square twisted bars were drawn upon the diagram. The intersections of these with curve AB indicated the point to which any set of bars would have to be carried. In order to fully supply resistance to diagonal tension, it was considered necessary to extend any size of bar above the point so determined by an amount equal to the distance between the centroids of tension and compression. This quantity was found to be approximately constant at varying depths at a value of 14 ft. Curve CD is therefore drawn 14 ft. from AB and parallel thereto. The actual arrangement of steel adopted is shown above CD and is self-evident. Connection between bars was made by right and left sleeve nuts screwed upon upset ends of the twisted bars.

In determining the amount of steel needed to carry shear, it was first assumed that the concrete of the caisson could carry a maximum stress of 40 lbs. per square inch upon its cross section. The remainder of the shear was to be carried by the steel at a stress of 10,000 lbs. per square inch. These bars were treated as stirrups in a circular beam and the spacing of them was determined by a modification of the formulas found in Taylor and Thompson's "Concrete," 2nd ed., p. 773. "Hoops" of 1 in. square twisted bars were placed inside and outside the vertical steel, the spacing varying from 6 to 12 ins. from center to center.

A Self-Contained Gasoline Rock Drill.

(Contributed.)

The field of air and steam rock drilling has been invaded by a drill using gasoline as the agency of motive force. It is an entirely self-contained drilling unit, consisting only of the drill and tripod, and requires no other equipment. Mr. John V. Rice, Jr., of Philadelphia, is the inventor, and it is the outcome of his many years' association with the rock drill industry.

The drill has a full floating, free piston action. The piston rod, pistons and bit are all connected together, forming a direct striking piece, acting and striking the rock in the same manner as the air and steam drills. The motive force is furnished by two cylinders opposed to each other, Fig. 1. A piston rod, fitted with a piston for each cylinder and on the end of which is the bit chuck, continues through the two cylinders. A make and break spark plug for each cylinder are tripped and fired by each piston alternately. The firing of the rear cylinder drives the piston and bit forward, hitting the rock directly with all the force of an explosion of gasoline. The front cylinder then fires and the piston is sent back to its first position again with the full force of an explosion, lifting the bit with extraordinary power. This operation continues indefinitely.

The pulsator is of the two cycle design (no valves used) and is fired at each stroke of the piston. The cylinders are water cooled under the control of a self-contained pump operating simultaneously with the drill. The fuel and oil container is attached to the side of the drill and is adjustable with respect to the angle at which the drill is set. When running, the lubrication of the working parts is automatic. The oil is fed into the gasoline and the cylinders, pistons and bearings receive the amount necessary.

The drill strikes 600 blows a minute with a bit 24-ft. long and drills holes from 1⅛ ins. to 2¾ ins. in diameter at any angle. The cutting speed is under the control of the operator, but when running at full speed the drilling is as rapid as the air or steam drills of equal cylinder size and weight. It operates for 10 hours on less than 3 gals. of gasoline when drilling continuously at the highest speed. The cost of daily operation is estimated at $3.50 for drill runner, $2.50 for drill helper, 75 cts. for gasoline and 10 cts. for lubrication, a total of $6.85. The makers are the Rice Gasoline Rock Drill Co., 1530 Land Title Building, Philadelphia, Pa.

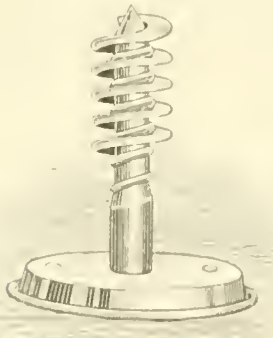


Fig. 1. Screw Anchor with Countersunk Clamp.

Method of Placing Concrete in Lining a Mine Shaft.

The lining of the No. 2 Hamilton shaft of the Chapin Mine at Twin Mountain, Mich., furnishes a suggestion to engineers having vertical shafts to line in the matter of form work and concrete placing. The excavated section of the original shaft was 10x24⅓ ft. and it was 1,434 ft. deep from collar to bottom; it was originally lined with timber. The relining involved removing the old timber and constructing new concrete lining and compartment framing. Figure 1 indicates quite clearly the old and the new construction.

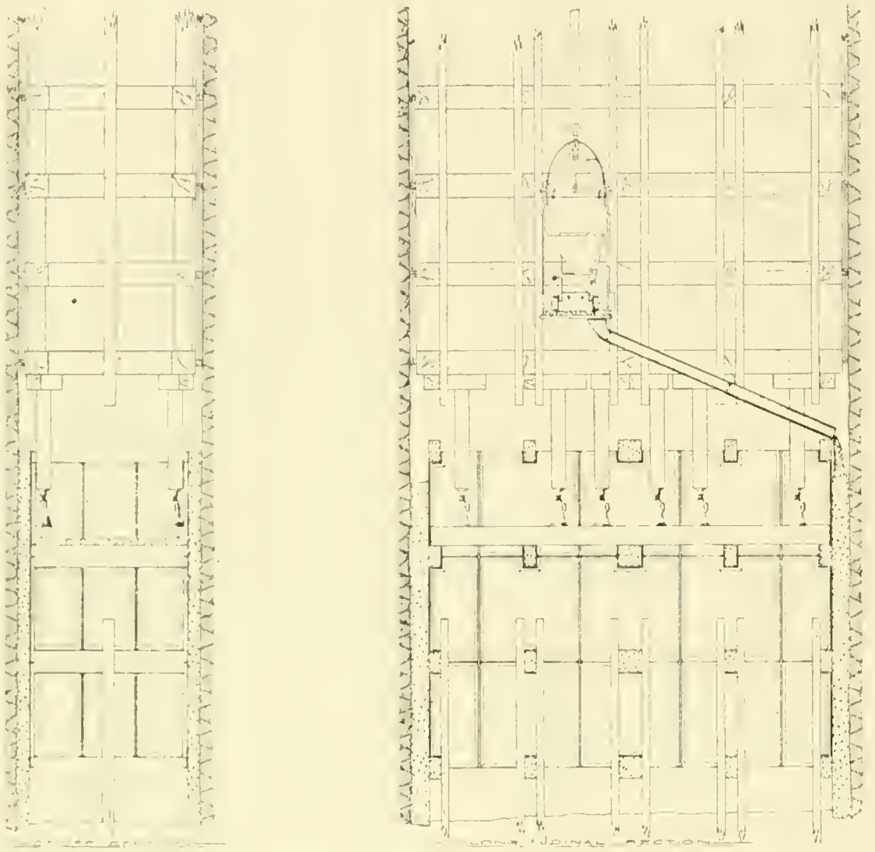


Fig. 1. Sections of Mine Shaft Showing Method of Relining with Concrete.

The general procedure was to place the lining in sections starting down shaft and working upward to a key or closure. Thus the first section was begun 83⅓ ft. below the top and closed at the collar; the second section was begun 219⅓ ft. below and closed at elevation 83⅓ ft., and other sections of varying height followed in order. In beginning a section, timber bracers were divided to support the old lining. A section of old timber, usually 12 ft., was removed and the timber above was held by posts on jack screws sitting on 12x12 in. timbers placed on the bracers. After the first 6-ft. section of concrete was placed the 12x12 in. timbers were placed as shown by Fig. 1 in the reinforced concrete dividers A, B, C and the end plates D, E. Here we may note that the dividers and end plates were separated with bracers moulded on the surface.

The lining forms as indicated by Fig. 1 were flanged steel panels with suitable notches or recesses for the ends of the concrete divider beams and end plates. A set of panels forming a 6-ft. high ring around the shaft was set up and butted together. Next the dividers and end plates were set in place and butted to the steel panels; these struts braced the wall forms firmly. Next the wall reinforcing was placed. The work was then ready for concreting the walls.

Two cages operated in the shaft. The concrete was mixed at the plant on the surface, using Smith mixers, which discharged into side dump cars. These cars were run into either cage and lowered to the work, where a revolving chute, as shown by Fig. 1, was attached to the car discharge spout and the concrete chuted behind the forms. All crevices in the shaft walls were packed to within 10 ins. of the forms with stone before the concrete was poured. The average amount of material for 6-ft. vertical section of shaft was 10 cu. yds. of concrete, 1 cord stone back filling and 550 lbs. reinforcing steel.

The force employed was as follows: For removing old lining, 1 man at the top and 5 men below; for placing steel forms, 2 men at the top and 4 men below; for pouring the concrete, 2 men at the top and 4 men below. In addition there was a shaft foreman, a concrete foreman, a, hoisting engineer, who gave part of their time as did the men working in the mining plant. Eight hour shifts were worked and the average time required to concrete one 6-ft. section was three shifts or 24 hours, which includes placing forms, removing one set of forms and concreting. The average rate of progress was 56.7 ft. per month, including delays.

The above description is condensed from a paper by S. W. Tarr in Proceedings, Lake Superior Mining Institute, 1913.

A Screw Insert for Anchor Bolts for Concrete Work.

(Contributed.)

Inserts that can be used in the forms have become an important detail in concrete structures because a little consideration in locating inserts on the plan does away with an immense amount of costly drilling when the structure is completed. Some architects stud factory ceilings and side walls with inserts at the corners of squares 2 or 3 ft. on a side over the entire area so that shafting and other fixtures may be readily attached at any time. These inserts are usually of cast iron and can be nailed to the forms; when the forms are removed the metal face of the insert shows at the surface of the concrete.

The illustrations show a steel spiral insert that has been on the market for some time, but the metal clamps for nailing the bolts to the forms are a new feature. The spiral anchorage can be located at any depth from the concrete surface, thereby giving any holding power up to the tensile strength of the bolt. The head of the bolt may be countersunk, as shown in Fig. 1, or flush with the surface, Fig. 2. The second method requires a shallow auger hole in the form; never more than 9/16 in. for a 1-in. bolt, so that no injury is done to the forms. Entire fixtures may be countersunk by using a clamp as large as the base of the fixture. Several bolts may also be accurately set as a unit, the clamp acting as a templet. The clamps are of thin metal and can readily be punched to order.

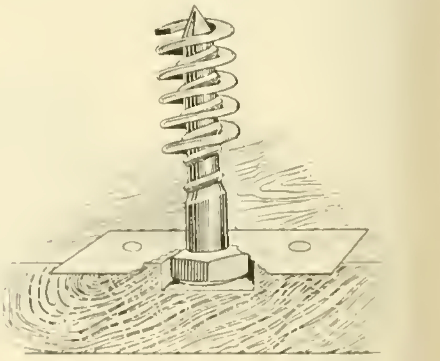


Fig. 2. Screw Anchor with Flush Clamp.

The small size of this insert makes it ideal for complex reinforcement, as it will readily fit between the reinforcing rods. These inserts have been extensively used on the Baltimore dock work. They are made by the Richmond Screw Anchor Co., Pulitzer Building, New York City.

Engineering and Contracting

Devoted to the Economics of Civil Engineering Design
and to Methods and Cost of Construction

Volume XLI. CHICAGO, ILL., MAY 6, 1914. Number 18.

Governor Glynn's Suggestion That Convicts Make Paving Brick; and Some Erroneous Inferences as to Cost of Macadam Maintenance.

Governor Glynn, of New York, has urged the state legislature to provide an appropriation for an experimental brick-making plant to be operated by convicts. The Governor further urges the use of brick exclusively for road work, and, to this end, he proposes to employ the state convicts as brickmakers.

Whatever merit exists in the idea of convict-made brick is likely to be overshadowed by the absurdities that accompany the plan of wholesale brick-making, wholesale abandonment of all types of road other than brick, and wholesale error as to the cost of macadam maintenance. The Governor says:

> On New York's 12,000 miles [when completed] of macadam roads the annual maintenance will be $12,000,000, the total cost of the roads will be $130,000,000, and at the end of ten years from the date of completion the state will have little to show for an expenditure of $250,000,000. In the twenty years thereafter the tax for the highways will be at least $20,000,000 each year if our roads are to be kept in proper condition.

Governor Glynn is less to blame for this nonsense than are the statistics of road maintenance that he has consulted. As we have editorially pointed out more than once, it is not a fact that the annual maintenance and depreciation of the average bituminous macadam road, or even the average water-bound macadam, is anything like as much as 10 per cent of its first cost. Governor Glynn estimates maintenance alone nearly 10 per cent and then adds 10 per cent for depreciation! The rebuilding of old water-bound macadam into bituminous macadam has been charged to "maintenance" in New York state, as well as in other states; and this has deceived many people into believing that the true maintenance costs will be as large as these seeming maintenance costs. We call them "seeming maintenance costs," for they really contain two elements beside true maintenance expenses, namely, (1) additions and improvements and (2) experimental construction.

When the advent of the automobile made it necessary to design new types of road, a very great amount of experimenting was begun on a large scale. It was not always called experimenting, but such it was nevertheless in many instances, and particularly in New York state. When an existing macadam road was scarified, stone added to increase its thickness—and often its width also—and bitumen added to bind the stone, all this was usually charged to "maintenance." But the major part of this cost was not true maintenance of an existing structure at all. As well charge the entire cost of a new concrete bridge to maintenance because it has replaced an old wood or iron bridge.

Not only have additions and improvements often been improperly charged to road maintenance, but nearly all experimental road work has been similarly charged. Now, to be perfectly candid, every highway engineer must concede that a very large part of all new road construction during the last few years has been experimental. To design a road that would be economic under both automobile and horse traffic was a new engineering problem, and it justified experimenting on a large scale. *Many a mile of new road has failed, and this failure has been improperly regarded as true depreciation.* Quite of a piece with such an error in accounting would it be to charge to incandescent lamp maintenance the entire cost of the research and inventions that led to the development of the mazda lamp.

Progress costs money—much money if there is to be much progress—but let us avoid the sophistry of charging the cost of progress to the *maintenance* of existing things.

Governor Glynn has been deceived, and many an engineer has been deceived, by the use of so-called maintenance expenses that were mainly costs of *additions* to existing things plus the costs of *experimenting*. To take one instance that comes to mind: A certain western county began extensive road construction a few years ago. Much experimenting was made in the use of asphalt and in the method of building asphalt macadam. Some roads remained good under traffic, but many failed. Out of this experience have come methods of building asphalt macadam roads that show no sign of failure after years of use, and upon which the expenditures for maintenance are nominal. Yet, if all the cost of rebuilding the roads that failed be charged to maintenance, it can be shown that the "maintenance" costs during those experimental years would be entirely prohibitive of further bituminous road work. It is just this sort of a condition that has deceived Governor Glynn, because it may not be interpreted properly by his advisers.

It is a matter wholly immaterial to ENGINEERING AND CONTRACTING what types of road shall survive. But it is important that both road builders and the public in general shall not be deceived by the statistics so commonly quoted to prove high costs of road maintenance. There is every likelihood, we believe, that many types of roads will continue to find favor. The economic conditions that make a brick road preferable in certain localities and under certain traffic will not necessarily exist in all localities and under all traffic densities. And the same can be said of every other type of road.

Governor Glynn's suggestion that convicts be employed in making paving brick is worthy of consideration. But let it be remembered that labor is among the least of the cost elements involved in vitrified brick making. The fuel for brick burning and the interest, maintenance and depreciation on the brick plant constitute the largest part of the cost of production. Convict labor is notoriously inefficient even where no skill is required. The manufacture of brick is a process requiring a great deal of skill, if the product is to be fit for use as a road surface. It would seem, therefore, that the manufacture of good vitrified brick is likely to result in economic failure if undertaken as suggested by Governor Glynn. Let it be proved that the convicts in New York state can economically produce the broken stone needed for the concrete foundation of the proposed brick roads before the state embarks on a manufacturing experiment of such doubtful success.

Foundations for Buildings Resting on Yielding Subsoils.

The greatest mistakes of designing are often made in connection with structures which do not appear to possess exceptional features. If the building is a very large one, or if it is a comparatively narrow one of great height, we are inclined to give each feature of its design careful consideration, but for the average structure we are apt to neglect details which later may prove to be very essential ones. The problem of proportioning accurately the foundations for comparatively low buildings resting on yielding subsoils is one which generally receives too little attention. It is essential that the exact character of the underlying strata be determined before starting the design of the foundation, yet many buildings are designed and their construction is undertaken without a careful survey of the soil conditons. It is not an uncommon occurrence for designers to find it necessary to modify the foundation design, even after construction is started. Soil strata often shelve rapidly, and a thorough knowledge of soil conditions may lead to the use of different types of foundations for various parts of the building if the best results are to be obtained.

In designing the foundations for comparatively low commercial buildings and for residences, where the substructure design is often considered relatively unimportant, especially by the architect whose attention is centered on the aesthetic design of the superstructure, poor judgment is often exercised. For such buildings the essential requirement of the foundation design is a uniform settlement of all parts of the building. We are so accustomed to seeing diagonal cracks in brick and concrete walls, and broken lintels, that we consider such defects incidental to building construction. Much of this unequal settlement could be obviated if engineers and architects would give more attention to the conditon of the subsoil, to the determination of the actual weight of the building and the distribution of this weight to the footings; and to the accurate proportioning of the footings for these loads. The best results are usually obtained by proportioning the footings for dead load only, using unit bearing values which are sufficiently low as not to be exceeded when the maximum live load is on the structure. We have known of cases where the footing loads were computed (superficially) from the preliminary architectural drawings, and were not even checked when the final drawings were completed. Any one familiar with building design will agree that there is usually a vast difference between preliminary and final architectural designs; and it is essential that the accepted footing design be based upon loads carefully computed from the final design drawings.

If the building is founded on rock or on a good quality of gravel it is unnecessary to use refinement in the calculation of footing loads, as the question of settlement is not such an important one, but for buildings resting on yielding subsoils we believe that this part of the design is not given sufficient attention. Furthermore, there is probably no part of the design where extra time can be spent to better advantage than in the careful proportioning of footings.

Tunneling for Water Supply Intake at St. Louis.

A very fully detailed article on the construction plant and methods employed in driving the new water works intake tunnel at St. Louis is published in the water works section of this issue. The shaft sinking operations are also described in detail. The article is of great value to all who are interested in rock tunneling.

The routine construction operations are enumerated and fully discussed for the purpose of indicating the relative efficacy of the different methods employed on the several major subdivisions of the work. Thus the handling of the plant, the drilling and shooting of the heading, and the handling of the spoil are all described so as to indicate the most successful methods devised for performing these operations on the St. Louis tunnel. The progress made in proceeding according to the several methods employed, particularly in handling the construction forces, is so recorded that the reader can readily form his own estimate of their comparative worth.

The organization of the forces, both on top and in the tunnel, is given and the rates of wages in effect are stated. The rates employed in the application of the bonus system, which helped materially in the progress of the work, are also stated.

General

Results of Tests to Determine the Action of Sea Water on Concrete.

There has been considerable discussion as to whether chemical action or frost action was mainly responsible for the disintegration of concrete constructed in salt water. To obtain data on this subject the Aberthaw Construction Co. of Boston has made a series of tests on 24 concrete piers. These piers were 16 ins. square and about 16 ft. long. They were placed in the Charlestown Navy Yard in 1909, and have been subjected to a mean tidal rise and fall of about 10 ft. The piers were placed with about 18 ins. of their length out of water at mean high tide; while at mean low tide only about 4½ ft. of their length were submerged. The tests were recorded by an officer of the Government navy yard, while careful analyses of the concrete and of the materials entering into its composition were made by Herbert L. Sherman, of Boston. To determine the effect upon the concrete piers they were inspected from time to time, the last inspection being made in December, 1913. The following data have been taken from the report of the tests made by the Aberthaw company.

In general, the specimens which were richest in concrete gave the best results. Of the 24 piers tested only 1 showed a disintegration of the entire section. This specimen was the leanest one tested, the concrete consisting of 1 part cement, 3 parts sand, and 6 parts broken stone, cast quite dry. The disintegration of the cross-section of this pier was complete at a distance of about 3½ ft. above low water. About 1 ft. of the length was entirely gone, the two sections being held together by the two reinforcing rods. The concrete was very slightly affected below the low water level. At the disintegrated portion the concrete was "dead," and could be picked off with the fingernails.

In contrast to this specimen, a pier composed of 1 part cement, 1 part sand and 2 parts broken stone, cast very wet, was in good condition after the five-year test. The concrete for this pier when cast was soft enough to run quite readily, but it would not flow like syrup. When examined in March, 1910, this pier was apparently in as good condition as when placed. When last examined the face and back were slightly pitted, but the edges and sides were in excellent condition.

CONCRETE MATERIALS.

Broken Stone.—The stone used for the concrete was broken trap rock, from quaries at Malden, Mass. It weighed 90.24 lbs. per cubic foot, and contained 50 per cent of voids. The gradation of size of stones is shown in the following table:

Diameter of ring, inches.	Percentage passing through ring.
⅛	0.25
3/16	0.50
¼	1.75
⅜	9.75
½	25.25
¾	53.00
1	80.75
1½	100.00

Sand.—The sand was obtained from banks at South Acton, Mass. It was clean and sharp, and was somewhat finer than that ordinarily used. When dry, its average weight was 97.18 lbs. per cubic foot, and it contained 34.2 per cent of voids. In the condition in which it was used it contained 4.6 per cent of water. Its fineness is indicated by the percentages given in Table II.

TABLE I.—COMPOSITION OF CONCRETE, WEIGHT PER CUBIC FOOT, CONDITION OF CONCRETE WHEN CAST, AND CONDITION OF SPECIMENS AFTER TEST.

Ref. No. of pier.	Concrete proportions. Cement.	Sand.	Stone.	Quantity of water in pier, lbs.	Net wt. of concrete, lbs. per cu. ft.	Condition of concrete and other data pertaining to ingredients.	Condition of pier; (a) on March 3, 1910; (b) on Dec. 17, 1913.
1	1	1	2	248	152.10	Dry.	(a) Front face slightly eroded; max. at half tide. (b) Face badly eroded for full length; sides and back in good condition.
2	1	1	2	295	151.06	Plastic.	(a) In excellent condition. (b) Face, sides and back slightly pitted below mid-tide.
3	1	1	2	425	145.04	Very wet.	(a) In excellent condition. (b) Face somewhat pitted; edges, sides and back in good condition, slightly pitted.
4	1	2½	4½	165	145.82	Dry—About 10 per cent excess of cement over voids, same excess of mortar.	(a) Extreme erosion about 1 in.; stones exposed on front face. (b) Face badly eroded throughout; back in good condition; sides in fair condition.
5	1	2½	4½	195	150.30	Plastic—About 10 per cent excess of cement over voids, same excess of mortar.	(a) In excellent condition. (b) Slightly pitted on all sides from top to bottom.
6	1	2½	4½	345	152.20	Very wet—About 10 per cent excess of cement over voids, same excess of mortar.	(a) Surface somewhat soft. (b) Sides in good condition; face somewhat pitted; corners eroded at low tide.
7	1	3	6	182	142.50	Dry.	(a) Sides slightly pitted; front face eroded, exposing stones. (b) Pier entirely eroded through at about 3½ ft. above low water (See Fig. 1, No. 7.)
8	1	3	6	240	146.58	Plastic.	(a) In excellent condition. (b) Front and sides pitted; back badly eroded from low to mid-tide, max. erosion 5 ins.
9	1	3	6	343	145.40	Very wet.	(a) In excellent condition. (b) In fair condition; face slightly pitted; edges somewhat eroded.
10	1	1	2	305	142.10	Wet—"Blanc" cement—Free from iron.	(a) In excellent condition. (b) In good condition; shows little bad effect.
11	1	3	6	353	146.20	Wet—"Blanc" cement—Free from iron.	(a) In excellent condition. (b) In good condition; slightly pitted below low water; edges fairly good.
12	1	1	2	325	145.90	Wet—"Atlas" cement—High in alumina.	(a) Slightly eroded on front face. (b) In good condition; front slightly pitted; edges all good.
13	1	3	6	345	142.80	Wet—"Atlas" cement—High in alumina.	(a) Slightly eroded on front face. (b) In fair condition; sides and back in good condition; face and one corner somewhat eroded
14	1	1	2	330	144.20	Wet—"Helderberg" and "Lehigh" mixed—Low in alumina.	(a) In excellent condition. (b) Face a little pitted; in good condition.
15	1	3	6	335	142.70	Wet—"Helderberg" and "Lehigh" mixed—Low in alumina.	(a) Slightly eroded on front. (b) Condition good for entire length; face pitted.
16	1	1	2	315	142.70	Wet—Iron ore cement, practically free from alumina.	(a) In excellent condition. (b) In good condition all over.
17	1	3	6	293	145.90	Wet—Iron ore cement, practically free from alumina.	(a) In excellent condition. (b) All four sides somewhat eroded; deepest holes ¼ in.; most erosion at low tide.
18	1	1	2	330	145.60	Wet—Universal cement (slag cement).	(a) In excellent condition. (b) In good condition; back slightly pitted.
19	1	3	6	370	143.30	Wet—Universal cement (slag cement).	(a) Slight roughness on front face. (b) Eroded badly between high and low water; corners gone; back eroded all over to max. of 4 ins.
20	1	3	6	275	145.50	Quite wet, exceptionally well mixed.	(a) Slight roughness on front face. (b) Slightly flaked on all four sides between low and half way to high water; corners slightly eroded.
21	This pier was lost.						
22	1	3	6	326	146.70	Quite wet—0.1 hydrated lime and 0.9 Portland cement.	(a) In good condition. (b) Face badly pitted; eroded 4 ins. at mid-tide; sides and back pitted.
23	1	3	6	298	137.90	Quite wet—Sylvester mortar—solution of soap and alum.	(a) Corners eroded in places. (b) Face badly eroded just above low-tide; corners at bottom gone; not much erosion above mid-tide.
24	1	3	6	275	151.60	Quite wet—5 per cent pulverized clay, 95 per cent Portland cement.	(a) Outer face somewhat soft. (b) Surfaces somewhat eroded between high and low-tide; corners eroded throughout; face worse than sides.

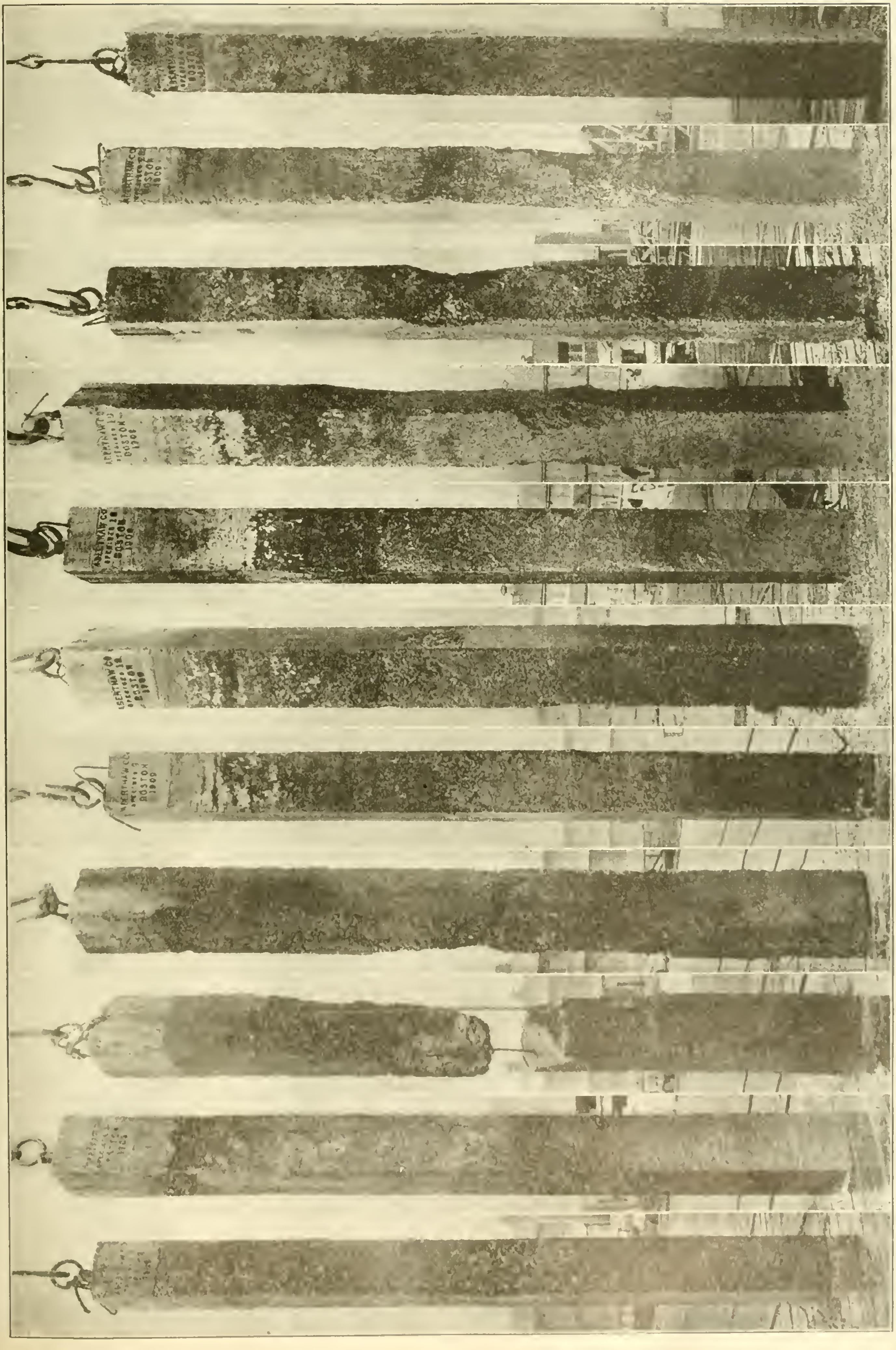


Fig. 1. Condition of Eleven of the Concrete Test Piers After Submergence in Boston Harbor for About Five Years. (See Table I.)

TABLE 11—SIEVE ANALYSIS OF SAND.

Sieve number.	Percentage passing through sieve.
100	4.8
50	24.0
30	52.0
20	73.4
16	86.4
10	91.0
8	93.4
6	96.1
¼-in. mesh	100.00

Cement.—Specimens Nos. 1 to 9, inclusive (see Table 1), contained commercial Portland cement possessing no particular characteristics. Specimens Nos. 10 and 11 contained "Blanc" cement; Nos. 12 and 13 contained "Atlas" cement; Nos. 14 and 15 contained a mixture of "Helderberg" and "Lehigh" cements; Nos. 16 and 17 contained an iron-ore cement; Nos. 18 and 19 contained "Universal" cement; No. 22 contained a mixture, consisting of 0.9 cement and 0.1 hydrated lime; No. 23 contained "Sylvester" mortar, a solution containing soap and alum; and No. 24 contained 95 per cent of Portland cement and 5 per cent of pulverized clay.

CONCRETE PROPORTIONS.

The concrete proportions for each pier are given in Table I. The quantity of water used, the net weight of the concrete per cubic foot and other essential data are also given in this table. Each pier was practically 16 ins. square and 16 ft. long. Pier No. 21 contained sea water instead of fresh water, but this pier was lost.

CASTING OF PILES AND GENERAL DATA ON RESULTS OF TESTS.

The piers were cast in a horizontal position, during the period Jan. 8 to Jan. 16, 1909, in one of the navy yard buildings. The temperature of the building was not very high, but it was kept above freezing. The forms were removed two or three days later, and the piers were all immersed in salt water between Feb 23 and March 1, 1909.

To give each pier sufficient stiffness for handling, a ⅝-in. square twisted bar, bent in the shape of the letter "U" (the legs of which formed reinforcement for the full length of the pier), was embedded in the concrete. The loop portion of the "U" projected a short distance above the top of the pier to enable a hook to be inserted. Later on (in November, 1913) it was found necessary to move the specimens to another location. The loops were found to be weak, and to facilitate the handling of the piers holes were drilled in their tops, and hooks were inserted in these holes and grouted in place.

A steel pipe, 2½ ins. in diameter and 8 ft. long, was embedded in the upper end of each specimen. This pipe was removed, leaving a hole in the specimen by means of which the percolation of sea water was measured. A short section of 3-in. pipe, with a removable cap, was fitted into the top of this hole to keep out the rain and splashings. The hole in each pier was sounded on Dec. 22, 1909, to determine the amount of sea water percolation. Sixteen of the holes were dry, while the others contained from 19 to 65 ins. of water. The depths of water in several of the piers were as follows: No. 7, 19 ins.; No. 8, 22 ins.; No. 15, 27 ins.; No. 1, 38 ins.; No. 5, 52 ins.; No. 4, 60 ins., and No. 2, 65 ins.

Table 1 (p. 516) gives data describing the composition of the concrete for each pier, the condition of the concrete when cast, and the condition of the pier on March 3, 1910, and on Dec. 17, 1913. As has already been mentioned the piers were placed in the sea water during the period Feb. 23-March 1, 1909.

Figure 1 (p. 516) shows views of 11 of the specimens. These views indicate the condition of these piers on Dec. 17, 1913. As will be noted by referring to Table 1, most of the specimens not shown in Fig. 1 were in good condition at the end of the test.

Design Features of 300-Ft. Steel Towers for Radiotelegraph Service at Key West, Fla.

(Staff Article.)

The Government is now having constructed at Key West, Fla., three 300-ft. steel towers for its radiotelegraph station at that point.

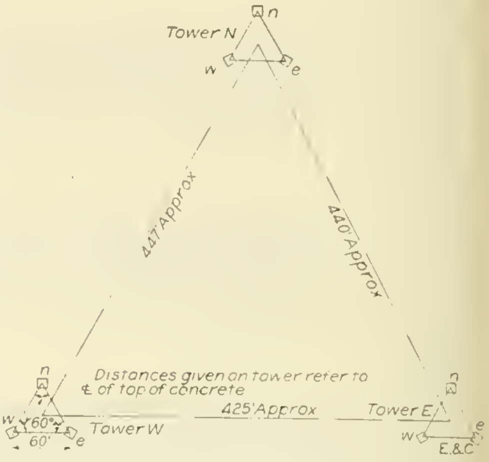


Fig. 1. Lay-out of Three-Tower Installation for Radiotelegraph Service at Key West, Florida.

TABLE I.—ELEVATIONS OF TOP OF PIER, GROUND AND ROCK FOR EACH PIER.

	Pier n.			Pier e.			Pier w.		
	Elev. top of concrete.	Elev. ground.	Elev. rock.	Elev. top of concrete.	Elev. ground.	Elev. rock.	Elev. top of concrete.	Elev. ground.	Elev. rock.
Tower N.	128.8	128.2	108.6	128.8	128.3	117.0	128.8	128.2	112.9
Tower E.	131.4	130.5	129.6	131.4	130.9	129.3	131.4	130.6	128.6
Tower W.	129.3	128.8	122.5	129.3	128.8	123.2	129.3	128.5	122.5

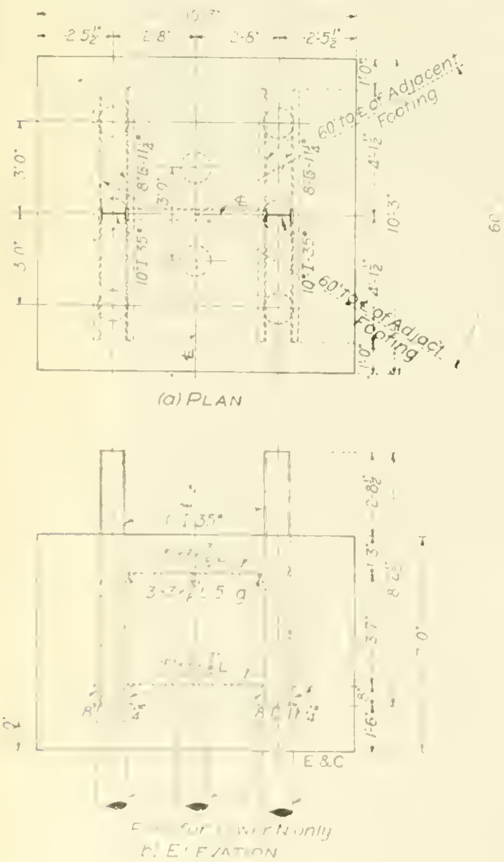


Fig. 2. Plan and Elevation of Piers for Tower Legs.

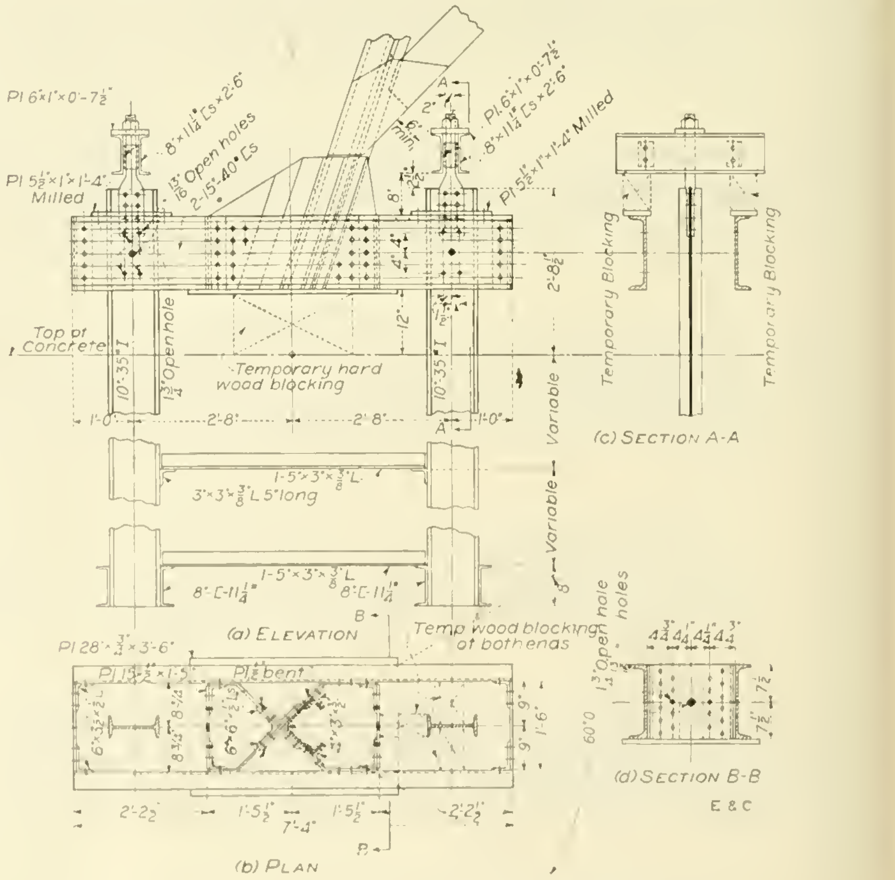


Fig. 3. Plan and Elevations of Anchorage for Tower Legs.

The base of each tower forms an equilateral triangle, with side lengths of 60 ft. Each tower leg is supported on and is anchored to a concrete pier resting either on a pile

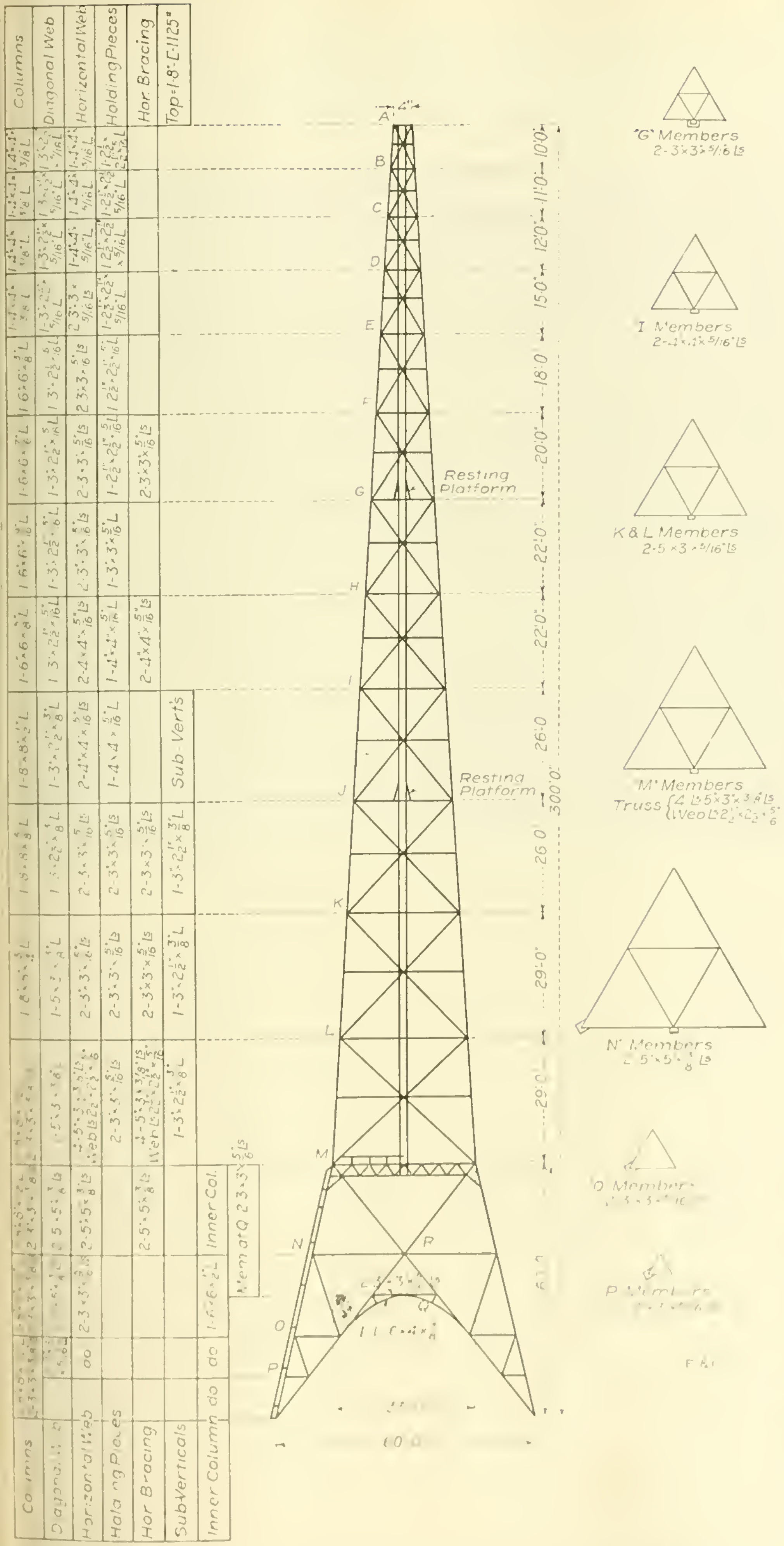


Fig. 4. Stress Sheet for 300-Ft. Steel Towers, Showing Principal Dimensions and Composition of Members.

foundation or on rock. A steel ladder is provided for reaching the top of the tower, where the sheaves for supporting the wire rope aerial are placed. The foundations were constructed and the anchorage was set by the Government, it having been specified that the steel for the latter was to be furnished by the contractor within 40 days after the award of the contract. In constructing the towers it was specified that the contractor could obtain electric current from the navy yard supply and could use the railroad tracks and rolling stock of the yard system and yard hoisting appliances, paying for the material and service at prevailing Government rates. It was further specified that the Government would furnish and install 1 meter, 1 switch, 1 transformer for electric current, in case alternating current was provided, and 1 water meter and shut-off each for fresh and salt water mains within the area assigned to the contractor. The structural steelwork was governed by the "Specifications for Mill and Foundry Products" issued by the Navy Department. The estimated weight of one tower is 50 tons and the estimated cost of the three towers is $15,000.

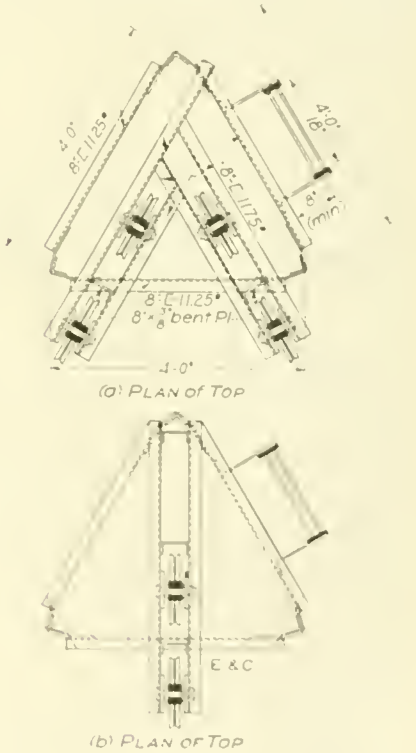


Fig. 5. Plans of Tops of Towers for Three-Tower and for Two-Tower Installations.

LOADS AND ALLOWABLE STRESSES

In designing the towers and their foundations a wind pressure of 30 lbs. per square foot of tower surface was used. The maximum resultant at the top of the tower due to the pull of the antennae, with this wind pressure, is 3,300 lbs.

The compression members of the tower were designed for the combined dead and wind loads, using an allowable unit stress of $18,000-70\frac{l}{r}$ where l is the unsupported length, in inches, and r is the least radius of gyration. The limiting value of $\frac{l}{r}$ used for main members was 100, and for secondary members 110. The allowable unit tensile stress was 18,000 lbs.

THE FOUNDATIONS

Figure 1 shows a lay-out of the three towers and gives the distances between the centers of the towers, together with the reference letter of each pier. The concrete piers for tower "N" are on pile foundations. Piling was not used for the piers of the other two towers, as the distance to rock was not great. Table 1 shows the elevation of the top of pier, the ground elevation, and the elevation of underlying rock for the various piers of the three towers.

Figure 2 shows a plan and elevation of a typical pier for tower "N". The pier supporting each leg of this tower is at an apex of an equilateral triangle with side lengths of 60 ft. Each pier is 10 ft. 3 ins. square by 7 ft. deep and is supported on 8 wooden piles driven to rock. The embedded anchorage for the tower leg is shown on the plan and elevation.

The piers which support the legs of tower "W" are of similar construction to that shown in Fig. 2, except that these piers rest on solid rock. By referring to the data given in Table 1 for these piers it will be seen that a small amount of rock excavation is necessary to obtain the required depth of 7 ft.

By referring to Table I it will be seen that the rock ledge at tower "E" lies only a short distance below the ground surface. For this tower it was specified that the depth of penetration might be reduced and suitable rods grouted into the ledge if the rock was sound.

ANCHORAGE FOR TOWER LEGS.

A detail of the anchorage for the tower legs is shown in Fig. 3. In general, the 10-in. 35-lb. I-beams extend 5 ft. 6 ins. into the concrete pier, although this distance varies for the piers of tower "E." It will be noted that temporary wood blocking is used under the tower legs. This is to provide space for the insulation which is to be furnished and installed by the Government.

SUPERSTRUCTURE.

The three 300-ft. towers are alike and are triangular in form. The length of a side of the triangle at the top is 4 ft. and at the bottom 60 ft. Figure 4 shows the stress sheet for these towers. The panel lengths of the towers vary considerably, as noted on the drawing, and are sub-divided as shown. The tabular data give the composition of each member. A ladder extends from top to bottom of the tower, and two intermediate resting platforms are provided.

Figure 5 (a) shows a plan of the top of the tower and gives the arrangement of the sheaves for a three-tower installation. The same towers are used for a two-tower installation, the arrangement of the sheaves being as shown in Fig. 5 (b). The following data apply to the sheaves:

	Ins.
Pitch diameter of sheaves	12
Flange diameter of sheaves	13⅝
Rim thickness	2
Diameter of pins	1⅜
Length of bearing	2½

Sheaves are cast-iron.

The towers are to be painted with three coats of paint, the shop coat consisting of the following proportions: 100 lbs. red lead, 20 lbs. white zinc, 5 gals. raw linseed oil, and 3¾ pints of turpentine Japan drier. It is specified that the steelwork is to be inspected after erection and all places where the shop coat is not in good condition are to be thoroughly wire-brushed and cleaned and given a further coat of red lead in oil. The two finishing coats of dark green paint shall have approximately the following composition: Zinc oxide, 45 per cent; sublimed white lead, 25 per cent; inert, 10 per cent; zinc chromate, 5 per cent; lampblack, 5 per cent; inhibitive Prussian blue, 5 per cent; and medium chrome yellow, 5 per cent; all well ground and of first-class manufacture, the paint to be prepared with raw linseed oil as a vehicle.

Water Works

Contractor's Plant and Methods Employed in Constructing the Tunnel and Shafts of the New St. Louis Water Works Intake at the Chain of Rocks.

Contributed by C. H. Hollingsworth, General Superintendent of Construction.

(The following article relates almost wholly to the construction of the tunnel, and shafs. The design of the tower, tunnel and screen chamber was fully described and illustrated in an article published in this section in the issue of Oct. 29, 1913. An article contributed by Mr. Hollingsworth, describing the construction plant and methods employed in placing the cofferdam for the new intake tower, which will also form a part of the works for the additional water supply, was published in this section in the issue of April 1, 1914.—Editors.)

The tunnel here considered is a part of the water works improvements now being carried on by the city of St. Louis. It runs from the new intake tower in the Mississippi River to the present pumping station at the Chain of Rocks plant. At the pumping station there is a large shaft for a screen chamber and a connection from this to the present pump well. From the screen chamber the main tunnel runs on a descending grade toward the river at about a 6 per cent grade for a distance of about 537 ft. to a drainage shaft. This short section of land tunnel has a sharp curve in it near the screen chamber. This curve has a 75-ft. radius and is 76 ft in length.

The drainage shaft is located close to the river bank and is about 96 ft. deep including a 6-ft. sump below tunnel grade. The sump of this shaft is connected by a new tunnel, 4 ft diameter inside the concrete lining, with an existing drainage shaft or well which was built some time ago to drain the old 7-ft tunnel with which it is connected by a small tunnel provided with a sluice gate. With the above arrangement after the work is completed either or both of the tunnels can be drained by opening the gates of the small drainage tunnels. The old drainage well is provided with a 10-in direct-connected centrifugal pump, run by an electric motor. From the drainage shaft the new tunnel runs out under the river for 2,253 ft. on an ascending grade of 1 ft in 1,000 ft and in a straight line to the new intake tower

All the tunnel work was done from the drainage shaft and the plant for the work was located there. The shaft is circular and 12 ft inside diameter in earth and 10 ft. in rock. As 32 ft. of the shaft is in earth and the bottom 14 ft. of that in quicksand a concrete caisson was used for sinking it down to rock. A pit was dug about 12 ft. deep and 18 to 20 ft. in diameter and a steel shoe made up of plates and angles was set in the bottom of the pit. On top of this the inside and the outside forms for the concrete lining, which was 20 ins. thick, were set up. The forms consisted of angle iron bent to the proper radius for ribs and 1 in. wood lagging. A view of the cutting edge and a portion of the form for the shaft caisson is shown in Fig. 1. They were set up in 5-ft. sections with the angles 3 ft. apart vertically. Concrete was then poured for the first 5-ft. section and the forms for the second section were placed immediately thereafter. By the time this section had been poured the forms could be removed from the first section and set up for the third section and so on to the top. The concrete was carried up for 20 ft. in this manner when all the forms were removed and the excavation started inside the caisson. Vertical timbers were placed against the outside of the concrete at four places around its circumference braced solidly, and plumbed up exactly to keep the caisson vertical. The progress through the lower 18 ft. of earth and sand was very rapid, as much as 8 ft. of shaft being excavated in an 8-hr. shift. When the caisson had reached the rock it was caulked with wooden wedges and oakum between the caisson and the rock. About 4 ft. of the shaft was then excavated in the rock and a collar concreted in it reaching from a point about 2½ ft. below the top of the rock to a point about 2½ ft. below it. This collar was about 11 ft. inside diameter and when the concrete lining in the rock section of the shaft is placed later on it will be 10 ft. inside diameter covering the collar completely as the lining will reach up to the top of the former.

Rock excavation in the shaft was started about Sept. 5, 1913, and four C-110 Ingersoll-Rand drills were used to do the drilling. They were mounted in pairs on two 4½-in. bars 10 ft. long, each bar being equipped with two arms. It was desired to do all the drilling on one shift and the mucking on the other two. This could be accomplished when water did not interfere with matters but as the work progressed water seams were often encountered which had to be grouted and that took a great deal of time.

The two bars were set up in the shaft as far apart as possible and all holes were drilled from the one setting. As a usual thing 6 or 7 ft. cut holes and 4 or 5 ft. side round holes were drilled. At the beginning of the drilling shift the gang would set up the machines and after drilling the complete round would shoot the cut or sump holes as they went off shift. The next gang would muck the cut-out clean, reload the cut holes if necessary and shoot it and the trimming holes and then muck out as much of that as possible. The third shift would then come on, clean up all the muck, reload any holes of which a foot or more remained, shoot those and then clean up ready for the drilling shift again. Working on this system and with even a fairly dry shaft a progress of from 4 to 6 ft. per day could be maintained. As it was, however, the progress from start to finish on the rock section of the shaft averaged about 2.6 ft. per day not allowing for lost time. Counting out the time lost in grouting, etc., the progress would run about 4 ft. per day.

When the shaft had been sunk to tunnel grade two shots were taken out of each tunnel heading after which the sump was shot out. Later when a short distance of tunnel had been driven each way the small drainage tunnel was started. The two main tunnel headings were each carried in about 200 ft. and the drainage tunnel about 50 ft. before the cages were installed. Previous to installing them the muck was handled in wooden box cars with a door in one side for dumping the load. These were the cars used for handling muck all through the work. The bottom of the car was covered with steel plate and had a 12-in. slope toward the door side of the box. End and front views of one of these cars are shown in Fig. 2. Some of the cars were temporarily fitted with a hook on each corner of the frame to which a four-way chain was hooked. The cars were loaded in the heading, run out to the shaft where they were picked up with the derrick by means of the chain mentioned and lifted to the top. They were there set down on a 21-in. gage track and run out and dumped.

For sinking the shaft round iron dump buckets were used, holding about ¾ cu. yd. These were handled by a stiff leg derrick with a 50-ft. boom and the spoil was dumped near the shaft for filling purposes. When ready for the cages the headings were shut down and a small gang was put to work cutting hitches for the shaft timbers. The sets of timber were placed about 8 ft. apart and were let into the rock at the corners. The timbering was of 6 x 8 in. pine; 2 x 6 in. planks dressed to fit the channels on the cages were used for slide rails for the latter. Above the rock the timbers were bevelled at the corners to fit the circular concrete shaft lining to which they were fastened with lag screws running into pine plugs driven in holes drilled in the concrete. There was just enough room for two 6 ft. 6 in. x 4 ft. 1 in. cages in the shaft but as the latter was circular there was space enough left outside the timbering on two sides for discharge pipes from pumps, air and steam lines, etc. A 4-in. air line was run from the compressor to the shaft and down to the bottom of the latter where it branched into two 3-in. lines, one running to each heading. To supply air a 900 cu. ft. per minute capacity two-stage Norwalk air compressor was installed with a 3 ft. x 14 ft. receiver. To supply steam for the compressor, cage hoist, and pumps, three 75-H. P. locomotive type boilers were used. A Webster, Camp & Lane single drum, link motion reversible hoisting engine with a 48 x 18-in. drum operated the cages. The latter were rove up double with a sheave on top of each cage. The cable for one cage led off the top of the drum of the engine, over a sheave on the head house, through the sheave on top of the cage and back up to the top timber of the head house. The cable for the other cage led off the bottom of the drum of the engine. A No. 2, Root reversible blower was installed in the house with the hoisting engine. From this blower a 10-in. pipe led to the top of the shaft where it branched into two 8-in. pipes. The latter led down the shaft one being for the east heading and one for the west. This blower pipe was made of 22 gage galvanized iron, both rivetted and soldered at all joints. The pipe was made up in 15 ft. lengths with light malleable flanges at each end and in joining the sections together heavy roofing paper was used for gaskets.

For handling muck cars from the headings to the shaft a 3-ton, 24-in. gage, General Electric Co. storage battery electric locomotive was used. This locomotive had two sets of batteries so that one set could be charging while the other was in use on the car. The batteries were changed every 8 hrs. As the locomotive was too large to be sent up on the cages and as the batteries were charged on top a special car was made to handle them. This car was built up as high as the floor of the battery box of the locomotive and was run alongside of the latter on a double track. The four trays of batteries, 44 cells in all, were then slid over to the flat car and the latter run on the cage and taken on top where it was run into the battery shed, the old set of batteries slid off to the charging platform, and the new set slid from the platform to the car and the latter then taken below again and the new set of batteries placed on the locomotive.

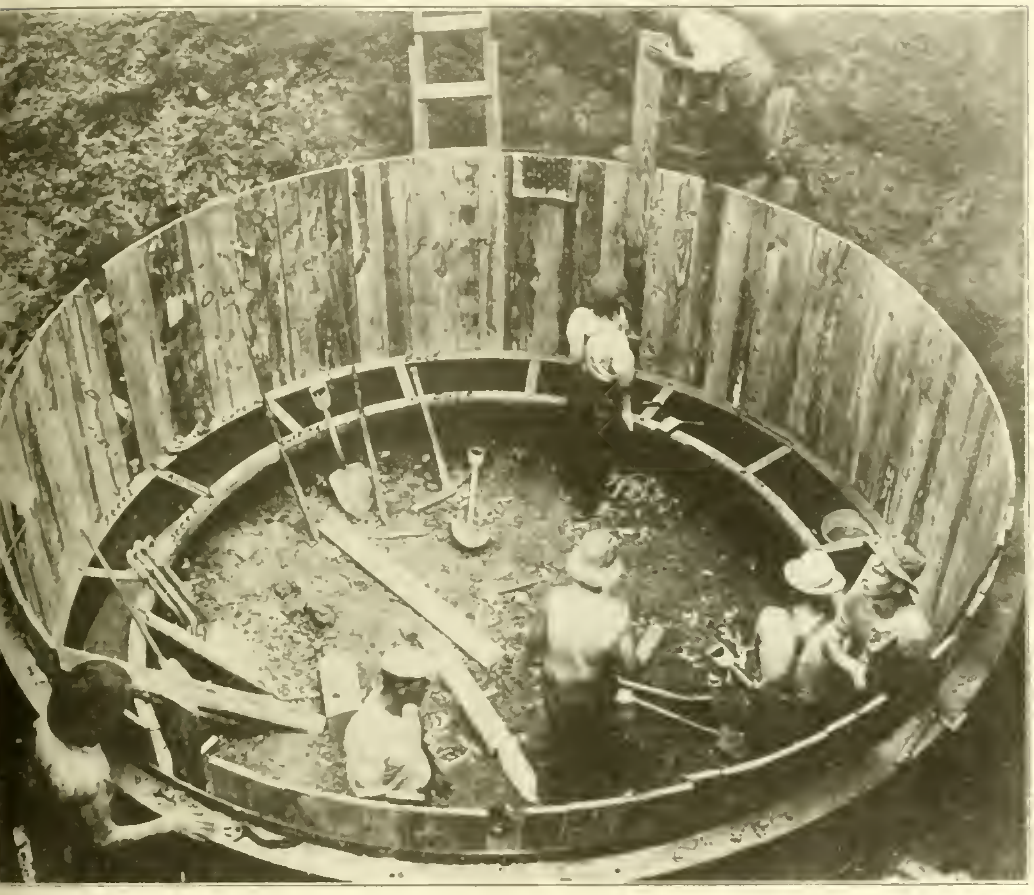


Fig. 1. View of Steel Cutting Edge of Shaft Caisson with Lower Section of Outside Form Complete—St. Louis Water Works Shaft and Tunnel Construction.

Electric current at 2,300 volts was obtained from the city and this was transformed to 220 and 110 volts for lighting purposes and for charging the batteries a standard Mercury Arc Rectifier was used taking 220-volt A. C. current on its primary side and delivering 110-volt D. C. current at its secondary side as the latter was what was used for the batteries. For running the Blower a 25 H. P., 550-volt D. C. motor was used and the current for this was obtained from the feeder line of the city's electric railway. This motor was reversible so that the blower could be run in either direction. The main feeder wires for the tunnel which were 0000, rubber covered, stranded cable, were carried from the transformer house to the shaft and down the shaft in a 2-in. iron pipe to keep them dry. From the bottom of the shaft where there was a switchboard a feeder ran into each heading.

A 3-in. steam line ran from the boilers to the shaft and down the latter to the bottom where connections were left for the pumps. The cage hoist was also tapped in on this line.

At the bottom of the shaft where the tunnels were enlarged slightly for that purpose, three pumps were installed, they being of 365, 350 and 150 gals. per minute capacity, respectively. The discharges from these pumps ran up to the top of the shaft where they were led into a 2-ft. square wooden culvert which discharged through the dump into the river.

At the top of the shaft a 24-in. gage track led west from each cage and ran into a single track about 30 ft. from the shaft. The single track ran into and terminated at the storage battery and transformer house. East of the shaft the two tracks from the cages ran into a single track about 20 ft. from the shaft and after about 6 ft. of single track it branched into two tracks, one of which swung abruptly south and the other north along the river bank and this afforded an excellent dump.

At the bottom of the shaft the tracks leading from the two cages toward the west heading came together about 12 ft. from the cages and continued into the heading as a single track. The tracks running toward the east heading came together about 12 ft. from the cages, continued for 6 ft. as a single track and then ran into a double track section about 100 ft. long, after which they continued into the heading as a single track. The first idea was to keep a switch close up to the heading so as to have a short section of double track next to the muck pile. In that way the empty and loaded cars could be switched around without upsetting them off the track. This scheme was found to be impossible, however, as the heading advanced so fast that it took up too much time keeping the switch moved ahead, which very much more than offset any gain made. Therefore three empty cars would be pushed in ahead of the locomotive and upset off the track on their sides, the track being kept close to one side of the tunnel for that purpose. The three loads were then hauled out and the empties placed on the track as needed, one at a time. Except right after the shots were fired when two or three cars could be loaded at once by spreading the gang out along the heading three cars was the number hauled by the locomotive as that number was more than enough to keep the gang busy until the next train of empties came in. About 2 ft. of loose

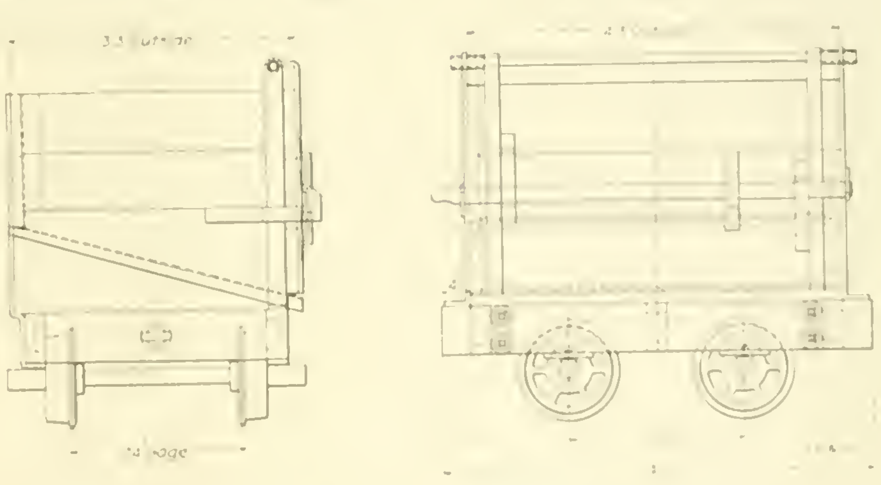


Fig. 2. End and Front View of Car Used in Handling Spoil from St. Louis Water Works Tunnel.

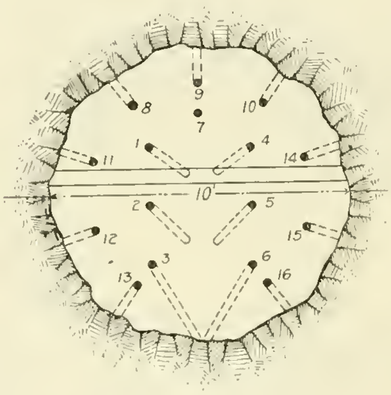
muck was left in the bottom of the tunnel so as to bring the track high enough to give width for two muck cars to pass each other. After the cages were installed gangs were worked in both the east and the west headings until the latter was finished on Dec. 17, 1913. Up to that time the progress was rather slow as the work was frequently interrupted to take sometimes a whole shift from the tunnel to work on the intake caisson for several days at a stretch. About Dec. 5, however, the gangs were shaken into place and for the week ending Dec. 12, the progress in the east heading was 102 ft. and the progress in the west heading was 86 ft. The latter was partly on a sharp curve which accounts for the difference between the two headings. One shot per shift was made or three shots in 24 hrs. and this was continued until Dec. 26, when a change was made to four shots in 24 hrs.

Meantime several methods of placing the holes had been tried. As usual, each of the heading foremen had a pet way of his own and each was different from the others, and on occasions the walking boss had still another. As I personally do not know of any fixed rule to adopt that will tell exactly how the heading should be drilled until you have found out just how the rock is going to break, I have during the past 12 or 14 years always experimented a little at first. In this case therefore I tried out each scheme that looked at all reasonable. Some of them looked very unreasonable, so we did not lose time trying them. It was remarkable how many ways could be suggested by parties who were doing what is technically termed the heavy looking on. In fact, I have never heard of so many ingenious ways of drilling a heading as were advanced at on time and another. Some of the usual schemes could not be worked owing to local conditions. For instance, a "hammer cut" could not be used as the bottom of the heading was round or rather pointed, and it was also impossible to use the old-fashioned "circular or star cut," as the muck pile was so high and so close to the face that the machines could not be lowered enough to point the lower holes of the cut, which also would have to be inclined upward. It was from the first a case of using the center cut but the proposition was to determine the placing and number of side round or trimming holes. Finally the gangs were started on the same method, which was to drill a center cut of 8 holes, 4 on each side, a breakdown hole above the cut, 1 center and 2 rib dry holes and 8 side round or trimming holes, making 20 holes in all. The bottom cut holes were inclined downwards, so as to break down to grade in the center. A sketch of the heading showing the location of the drill holes is shown in Fig. 3. Two 4½-in. columns 7 ft. long with double screws and with two arms on each were used for mounting the drills, which were the C-110 Ingersoll-Rand type.

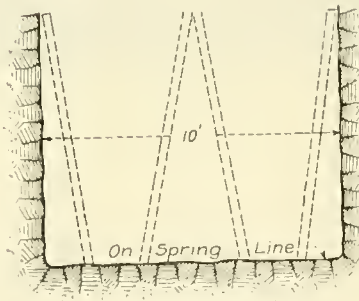
When getting three shots per day or one shot per shift the gang came on shift just after the previous shift had finished shooting. They mucked out, set up the columns and machines, drilled a round and shot it. The cut holes were drilled from 7 to 8 ft. deep and the other holes from 5 to 6 ft. deep. When possible, the two bottom holes were drilled about a foot deeper than the other cut holes, as they were drilled at such a downward angle that they would not bottom as far ahead as the other holes unless this was done. This would leave a toe that interfered with the drilling of the next round.

The rock encountered was generally a hard limestone with occasional streaks of soft limestone, which was usually brown in color. While fairly hard, the rock was the best possible sort to drill, as it was not so hard that the cuttings came out of the drill hole in large chips but came out or was washed out as a fine powder, but this advantage was lost on account of the numerous horizontal seams in the rock. It lay in beds from 3 ins. to 12 ins. thick, with occasionally one 2 or 3 ft. thick. As the seams were almost exactly horizontal and as none of the holes was level, at least one of the seams would be crossed by each of the holes and usually several. They would cross the seams at a very flat angle and as most of the seams had a little soft material in them the drills would strike the seams and run off at an angle and jam, thereby causing a lot of lost time. The rock also required a great deal more powder than if it had not been so seamy, as at times with holes only 2 ft. apart vertically a bed or two beds would be left in between them after shooting. The rock would break cleanly to the bottom of the holes, both above and below these beds, but they would remain undisturbed. To avoid this it was necessary either to use a lot more holes or to load the holes considerably heavier. The latter was the course followed. When sinking the shaft where the drilling was at right angles with the seams it required about 30 per cent less powder per lineal foot of hole and fewer holes were drilled.

In the tunnel several different kinds of powder were used. The 40 per cent Forcite gelatine dynamite gave fair results and very little more of it was required than of the 50 per cent or even the 60 per cent, but it made a great deal more smoke and gave trouble that way. The 60 per cent which was used at first was a little too quick in its action and the 50 per cent proved to be the happy medium. Another disadvantage of the seamy rock was that it was difficult to keep the roof arched to the circular tunnel section. A seam that was on or near the roof grade would be followed and the center dry hole would have to be bottomed high enough to break to this seam. If the two rib dry holes were bottomed too high they would shatter the rock so that it would later come down as far up as the roof seam, leaving a flat roof. If, on the other hand, the holes were kept too low the roof would break in the form of an inverted letter V and also the center dry hole would have too much work to do and would not break the ground properly. Also when the roof broke in that manner it was difficult to set up columns, and in such cases a 4½-in. shaft bar 10 ft. long with four arms on it was used. Figure 4 shows the manner in which the rock broke out under blasting.



Section of Face



Horizontal Section

Fig. 3. Section of Heading of St. Louis Water Works Tunnel, Showing Location of Drill Holes.

When the west heading had been completed on December 17 the best men from both headings and the best heading foremen were selected for work in the east heading. After they had gotten well organized it was found that there was considerable time to spare on each shift after they had finished their round of holes and shot them. It was impossible to increase the progress by drilling a longer round, for even with an 8 ft. cut the ground could not be shot to best advantage. This was because of the limited room in the tunnel in which to swing the machines, making it necessary to drill the cut holes at a very acute angle with each other. In practically all cases with an 8 ft. cut it was necessary to shoot the cut at least twice. The only way, therefore, to increase the progress was to increase the number of shots in 24 hours, and as the time was not ripe for two shots per shift a schedule of four shots per day was started on Dec. 26. The day shift came on at 8 a. m., after the previous shift had finished shooting, mucked out, set up, drilled a round and shot at 2 p. m. They then mucked out and set up ready for the next shift to start drilling. The 4 to 12 shift came on, drilled a round and shot it at 8 p. m. They then mucked out, set up and drilled from 10 to 12 holes of the second round before midnight. The 12 to 8 shift finished the round and shot at 2 a. m. They then mucked out, set up, drilled another round and shot at 8 a. m. About this time we also began to cut down on the number of holes drilled in the heading and finally got it down to 17 and 18 holes. Also in order to leave as little muck as possible at the face, two muck shots of about 7 to 10 lbs. of powder each were placed one on each side of the heading in the loose muck and fired with the last shot in the heading.

This threw the muck well back down the heading and left very little more mucking to do before the columns were set up. With three shots in 24 hours the progress for a seven day week would run about 102 ft. per week or about 14.5 ft. per day. After starting on four shots per day the progress ran up to 132 or 133 ft. per week or about 19 ft. per day. In a way, however, the four shots per day system never gave entire satisfaction, as each shift was leaving a round partly drilled, and no foreman could either set up or start a round to suit the following man, and there was always trouble on that account. Also the time of shooting on the 8 to 4 shift and on the 12 to 8 shift interfered seriously with the mucking. As soon as things were running fairly well with four shots per day I began to arrange for two shots per shift. To shorten up the drilling time the drill bits were cut down in size. The starter had been 2½ ins. on the bit and the 4, 6, 8 and 10 ft. were respectively 2¼ ins., 2 ins., 1¾ ins. and 1½ ins. The starters were made with a 2-in. bit and each succeeding drill was decreased ⅛ in., making the 10 ft. still 1½ ins. This did not work well, as the rock was hard enough to wear off the gage of the bits so that the next steel could not be pushed into the hole or, in other words, the drills would not follow. Also as the powder in use was 1⅛ ins. in diameter it was not necessary to bottom the holes larger than 1¼ ins. at the outside. By taking the bit of the 8-ft. steel as 1¼ ins. and increasing each size 3/16 in. over the preceding one the starter could be made 1 13/16-in., and if it was required at any time to use the 10-ft. drill it could be made with a 1⅛-in. bit and the hole would still be large enough for the powder if the latter was squeezed up and stripped of the paper. This made a very great saving in the drilling time. Another point that helped out was the carrying of both the air line and the blower line closer to the face of the heading. The horizontal seams in the rock made the muck very dead and it did not fly any great distance when it was shot. When this was ascertained the 8-in. blower line was run up to less than 75 ft. from the face, so that after a shot and with the blower drawing out the smoke, the heading would be clear in a few minutes. The 3-in. air line which supplied the air for the drills had previously been kept about 70 ft. back from the face. On the end of it when the drills were working was a 50-ft. length of 2-in. hose terminating in a 6-branch manifold from which a 25-ft. length of ¾-in. hose ran to each drill in the heading. When the round had been drilled the small hose was taken out with the drills and after the holes had been blown out the large hose was also carried back by the muck gang, and a 1-in. nozzle was screwed to the end of the 3-in. line. After the first shot was fired the blower was started and the valve on the air line partly opened. The compressor at the surface while the air was blowing in the heading would be run slowly, keeping the pressure on the gage at about 10 to 20 lbs. Even this pressure, with the aid of a 1-in. nozzle, would blow the air right up to the face, more especially as the men were going in after the first shot had been fired would block up the end of the pipe a foot or more. The 50 ft. length of 2-in. hose was shortened up to 25 ft. and this left the end of the air pipe back only 40 to 50 ft. from the face and made a material difference in the clearing out of the smoke. With the arrangement as above outlined the smoke could be cleared out in less than five minutes. In fact, when the heading was dry the men walked down to the shooting switch, a distance of about 600 ft., fired the shot and walked directly back into the heading, and even then would find the smoke cleared out as far back as the end of the blower pipe. Back of that, however, there would be considerable smoke, which would clear off in the course of half an hour. When there was water in the heading and coming in from seams along the tunnel near the face the smoke would hang for an hour or more, and even after the smoke was cleared out there would be considerable gas left and this latter never was noticed when the heading was dry. In handling the smoke the blower was first run so as to draw it out and after about an hour of that it would be reversed and clear air would be blown in.

On Jan. 21 six shots per day were started and the progress jumped to from 25 to 27 ft. per day. Each shift came on as the previous shift finished shooting, mucked out, set up, drilled a round and shot it in the first four hours of the shift and again in the last half of the shift. As a usual thing they drilled a 6 to 7 ft. cut and a 4 to 5 ft. side round, depending on how soon they got started drilling. The number of holes was cut down to 16, with occasionally a center lifting hole in the bottom when there was a toe left from the previous shot. This gave each driller 4 holes to drill and evened up the work. On account of trouble in setting up columns and water in the bottom of the heading the columns were discarded and the 10-ft. bar used. An arm with a jack screw in the end of it was placed in the center of the bar with the jack screw bearing against a block on the muck pile back of the bar. The jack screw in this arm was kept tight and this served to steady the bar while the drills were running. In shooting the heading the cut and breakdown holes were fired first, the latter being loaded with a delay action exploder. The cut was then reloaded, using instantaneous exploders, and it and the side round which was loaded with delay action exploders were fired together, making the second shot. Finally the dry holes and the muck shots were fired, making the third and last shot.

All shooting was done with a 220 volt A. C. lighting current from a switch which was kept from 600 to 800 ft. back from the face and on the opposite side of the tunnel from the lighting circuit to prevent accidents. From this switch a pair of No. 16 R. C. wires led to within about 50 ft. of the face, where they terminated in a coil of about 200 ft. of the wire at the start. As the heading advanced this coil was unwound until it had all been stretched out when the firing switch was advanced 200 ft. and 200 ft. of wire taken ahead and spliced to the end of the shooting line. From the shooting switch a short section of wire and a plug were used to plug in to a socket on the light circuit which had been left for that purpose. As this short section of wire stretched across the tunnel it was impossible to leave it in by mistake after the shooting was finished, as the motor car was continually passing and re-passing. When the powder was brought in to the heading the boxes were piled on the side of the tunnel opposite to the light wires. The exploders which were made up on top were brought in separately and piled separately from the powder. The nipper who came in with the exploders stayed near the powder and exploders until they were taken into the heading to load up. Before starting to load the light wires were taken back from 25 to 30 ft. at least.

On top, the powder and exploders were kept in magazines widely separated and no thawing of powder except by a slow warmth was allowed. When the powder was first delivered it was placed in the powder magazine, which was always kept at a temperature of 75 to 85°, and enough cases were opened up to provide powder for the next shift while the powder already opened would be used for the present shift. The powder as opened up was piled loosely on racks provided for that purpose. The steam radiator which furnished heat for the magazine was covered over with a screen down to within about 3 ins. of the floor. The screen had a sharply sloping top, so that no powder could be laid on the radiator or on the screen. There was room enough to sweep out under the screen. The powder was ordered well ahead, usually so that the supply very seldom went below 800 lbs. This gave the new lot of powder a chance to get partially thawed out before the cases were opened. It had 8 hrs. in which fully to thaw out after it had been opened. This was in extreme cases and as a usual thing the powder was fully thawed before the cases were opened

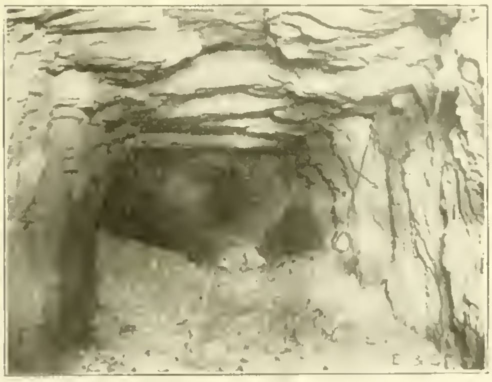


Fig. 4. View of Tunnel Heading After a Shot—Muck Shots Blew Out the Holes Where Men Are Sitting.

All drill steels were made and sharpened with a Leyner drill sharpener, there being one blacksmith and a helper on the day shift only. They handled from 120 to 200 pieces of steel per day and did all general work besides. In the blacksmith shop there was a coke forge for heating the drills, a coal forge for the other work and a D-24 Ingersoll drill, which was mounted on a column set firmly in the ground and which was used for a steam hammer.

MUCKING

The mucking was done at all times by a muck foreman and a gang of 6 to 8 muckers in each heading. With three shots per day five or six men were enough to handle the muck and keep the track in shape. The heading was so cramped, however, that it was a problem to get more men in it when the four and six shots per day systems were started. However, by carefully selecting the right and left hand shovelers and keeping them on their proper side of the tunnel and by training them to throw over their shoulders in place of turning all the way round, four men could work abreast ahead of the car to be loaded, while two more could work up on the muck pile, throwing over the heads of the other four as the latter stooped down for a shovelful of muck. These two men on the muck pile also loosened up the muck for the other four shovelers. In addition to those mentioned, one or two men could be worked behind the muck car, picking up bottom and cleaning up. It was also found necessary when getting six shots per day to have an extra gang consisting of a foreman and four to six muckers on the day shift to keep the track laid and the bottom ditched to carry off the water. In the heading ⅜-in. boiler plates were laid down before each shot, covering the bottom of the tunnel for its full width and back for a distance of about 25 ft. This made the shoveling easy and the muck cars could be pushed up on these plates beyond the end of the track to be loaded until a space long enough for a short section of track was cleared ahead of the rail ends. The two night shift muck gangs placed short sections of track temporarily and the day shift helped by the extra gang replaced these short sections with 30 ft. rails spiked down to 4x6-in. ties. The number of cars loaded in one 8 hour shift ran from 40 to 65 and towards the last it ran about 55 to 65 cars. As these cars held fully a yard, as loaded, and as the actual mucking time ran about 7 hours, it can be seen that there was no time to lose with the mucking.

SCREEN CHAMBER.

The work on the excavation for the screen chamber was started about Jan. 1, 1911. As a depth of 32 ft. of the excavation was in earth and the bottom 12 ft. of that in running sand it was considered advisable to use steel sheet piling. The screen chamber sets in an angle formed by the wall of the present screen chamber, meeting the wall of the pump well. For this reason the excavation in earth had to be handled with care and the timbering so done as not to allow the slightest movement. The rock excavation had also to be handled in a very careful manner, as it ran to a point 25 ft. below the level of the water in the old screen chamber and the pump well, which were separated from it by less than 10 ft. of rock on two sides. The only available steel piling was 26 ft. long and this was used, the tops being driven down level with the ground surface with the idea of setting them down to rock when the excavation had been carried down to their bottoms. The piles were driven before any excavation had been started and they were driven with a 3,000-lb. drop hammer. They went down very easily, except in one spot where several of the piles encountered some old timber cribwork. After the piles were driven a stiff leg derrick with a 50-ft. boom was set up and the earth excavation started. A standard gage railroad track ran across one end of the excavation and this was cut off and the stub end used on which to spot cars. The cars used were bottom dump gravel cars, holding 30 to 40 cu. yds. The earth and rock were dumped into these and hauled away for filling purposes.

When the excavation had reached a point about 4 ft. below the surface the first set of timbers was placed. It was 2 ft below the ground surface and consisted of 12x12 in. timbers around the outside braced across every 8 ft. with 8x12 in. and 10x12 in timbers. Other sets were placed as fast as the material was excavated, the timber gang working in one half of the shaft while the excavating gang worked in the other half. There was plenty of room for this, as the shaft was 10x32 ft in earth. Care was taken to tie the sets of timber together with 2x10 in lacing planks, spiked solidly to each set and to keep posts between the sets at every intersection. The top set was tied to the top of the sheet piling with hook bolts and the lower set was always kept blocked from the bottom. When the excavation reached the bottom of the piling it was not considered necessary to set the piling down. A short set of 2-in. plank sheeting carried the excavation to rock. There were seven sets of timbers required, the spacing between them being 6 ft. at the top and then 5 ft and then 4 ft. The rock surface was somewhat irregular and broken up but a good joint was made with the plank sheeting and about 4 ft. of the bed rock excavated over an area large enough to furnish a foundation for the entire screen chamber. After this the rock work was started. In the rock the shaft is 18x19 ft., with a channel 10 ft. wide and the full depth of the shaft swinging around on a curve from the shaft to the present wet well. The west tunnel heading, which was supposed to terminate at the screen chamber shaft, had been carried across the shaft to expedite the work in the latter. A round of holes was drilled upwards from this heading and shot, leaving the muck to be handled later. This left 12 ft. of rock between the top of the tunnel and the bottom of the shaft. One hole was drilled completely through from the top and shot a number of times until it had broken a hole through. One man with a Jackhamer drill was then put to work enlarging the hole, and from that time on this one man kept enough rock excavated to keep the muck gang of about 8 men busy. The shaft was first enlarged to full size and carried down to the bottom and then the channel was taken out toward the wet well. This latter was stopped at the outside of the wall of the wet well. The shooting had to be done very cautiously on account of the proximity of buildings and the short distance to the wet well and the old screen chamber. The work was completed on March 30, 1914.

GENERAL.

On the eight months' work there were no accidents with powder and no fatal or even serious accidents of any kind. About the worst accident was the breaking of one man's ankle.

The tunnel was too short in length and of a too cramped and unhandy cross-section to get up the maximum speed which could otherwise have been attained. In addition, the east heading under the river was on an extremely flat grade and gave a lot of trouble from water, even when the volume of the latter was not great, as it would back up in the heading and cover the track, making it hard to get the holes drilled in the bottom of the heading and putting the electric locomotive out of business, as it could not be run if there was even a few inches of water over the track. On the other hand, the grade in the west heading was over 6 per cent, going up toward the screen chamber from the shaft, and it was extremely difficult to handle loaded or even empty cars on that grade.

Also strong efforts were made to drive the tunnel as near as possible between 10 and 11 ft. in diameter and on cross sectioning the tunnel after it had been holed through it was found that barring the muck in the bottom it was slightly less than the 11 ft. cross-section.

ORGANIZATION AND WAGES.

In the regular gangs on each shift there were the following men at the rates given:

Force on Top: Number and grade.	Rate per day.
1 compressor engineer at	$7.00
1 hoisting engineer at	7.00
1 fireman at	3.20
1 signal man at	2.80
3 top men or car pushers at	2.40
Force in the Tunnel:	
1 pump man and pipe fitter at	4.00
2 cage men at	2.80
1 motor man at	4.00
1 switchman at	2.80
1 muck foreman at	4.80
6 to 8 muckers at	2.80
1 heading foreman at	5.00
4 drill runners at	3.60
4 or 5 helpers at	3.00
1 nipper at	3.00
1 electrician at	4.00
Force of extra men on Day Shift (on top):	
1 blacksmith at	5.00
1 helper at	4.00
1 machinist at	4.00
1 machinist helper at	2.40
In the Tunnel:	
1 extra muck foreman at	4.80
4 to 6 extra muckers at	2.80

RATE OF PROGRESS.

Taking all things into consideration, the progress was very satisfactory, especially while making the six shots per day.

During the month from 8 a. m. Jan. 21 to 8 a. m. Feb. 21 the progress was 741 7 ft. and for the best week, which was the week ending at 8 a. m. Feb. 20, the progress was 184 ft. This latter was for the full week of seven days or 21 shifts. The month's progress, however, was for 29 days and two shifts, as during the 31 days of the month there were four shifts lost time. The best progress made in one day was 29.1 ft.

The east heading was finished March 4, and a gang on each shift was started immediately cleaning up the bottom and trimming. The gang on each shift consisted of a foreman and ten men mucking, besides one drill runner and three helpers, with the usual cage men, top men, etc. On March 19 a second gang was started on each shift, there being a foreman and ten men in this second gang also. One of these gangs worked from the heading toward the shaft and the other worked from the shaft toward the heading, and on April 1 the trimming was completed and practically all of the bottom had been taken up. In places there were from 50 to 75 ft. that required no trimming at all and less than 200 ft. of the roof required trimming. On the other hand there was less than 100 ft. of the roof that was outside of the 11 ft. circle.

BONUS SYSTEM.

A bonus system was put in force in the early stages of the work and helped materially in the progress. A minimum of 72 ft. of tunnel per week was set and for all over that the following bonus rates were paid:

	Cts. per ft.
Heading foreman	50
Muck foreman	30
Drill runners	30
Drill helpers	20
Nipper	20
Muckers and cage men	10

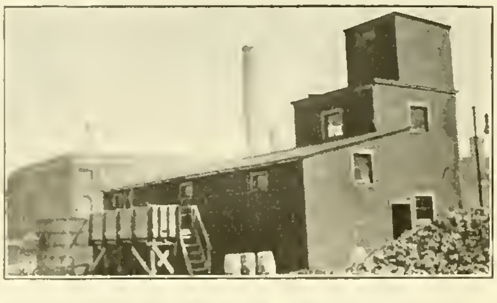


Fig. 1. Exterior View of Experimental Water Filtration Plant at Kirtland Pumping Station, Cleveland, Ohio.

From the total weekly progress ending every Friday morning at 8 o'clock 72 ft. was subtracted and each shift received bonus on one-third of the remainder, each shift being credited with one-third of the extra progress.

Throughout the entire work as described and including the design and installation of the construction plant C. H. Hollingsworth was in charge as superintendent for the Fruin-Colnon Contracting Co. of St. Louis, who were the contractors. The work is being carried out for water department of the city of St. Louis under E. E. Wall, water commissioner; Gurdon G. Black, chief engineer of the supply and purifying divsion; and E. C. Davis as resident manager.

Preliminary Work on the Cleveland, Ohio, Water Filtration Project.

Contributed by M. F. Stein, Assistant Engineer of Filter Plant Construction, Cleveland, O.

Owing to the increasing pollution of Lake Erie by sewage and the objectionable turbidity, especially following storms, the city of Cleveland, Ohio, is taking steps towards filtering its water supply. The pollution of the water necessitated the application of hypochlorite of lime as a precautionary measure, and popular prejudice against the tastes and odors resulting therefrom gave additional impetus to the filtration project.

In May, 1913, Mayor Newton C. Baker appointed a commission to investigate the feasibility of and to report upon the most practical means for filtering the city's water supply. The members of this commission are Prof. A. W. Smith, of the Case School of Applied Science, president; Mr. R. Winthro Pratt, consulting engineer and secretary; Dr Wm. T. Miller, member of the State Boar of Health; Prof. Hippolyte Gruener, of Western Reserve University, and Dr. R. G. Perkins, City Bacteriologist. In a report submitted in July, 1913, this commission recommended mechanical filtration as most practicable and advised the installation of a tes plant to investigate the problems incidenta to applying this process to Cleveland water.

To this end an experimental plant of 100,000 gals. per day capacity was installed, an has been in continuous operation. (An exterior view of this plant is shown in Fig. 1.) The experimental results obtained to dat have been of much value in furthering th cause of filtration as well as of assistance i the actual design of the plant. The uniforml clear and bacterially, free effluent of the plan is the most convincing argument to us against the views of opponents to the filtration scheme. The experiments have demonstrated the most suitable coagulants; th amounts required under varying conditions the period of subsidence necessary to th complete reaction of the chemicals, the settlin out of suspended matter and precipitate, an for obtaining the highest basin efficiency; th amount and nature of sludge obtained, etc Daily analyses of raw, settled and filtered water are made and complete plant records ar kept. It is hoped that these will be valuabl when the large plant is started, as affordin data on the proper treatment of Clevelan water, and that thus the mistakes commonl occurring in the early operation of a ne plant may be avoided.

Table I gives an average analysis of Lak Erie water as obtained from the intake tunnel of crib No. 3, four miles from shore. I will be noted that the total hardness is 11 p. p. m., which is principally temporary, th permanent hardness constituents being low Owing to the extensive use of city water fo manufacturing purposes (about 60 per cen of the total pumpage) it has seemed desirabl to soften the water. Experiments at the testing plant show that about 4.5 grains per gallon of 90 per cent lime are required for maximum reduction of temporary hardness As the precipitated calcium carbonate an magnesium hydroxid resulting have a ver marked coagulative action, only a smal amount of coagulant (about 0.5 gr. per gal. is required. Since the permanent hardnes is relatively low, and the use of sodium carbonate would tend toward foaming in boiler no attempt at its removal is made. It wi be noted that with the small amount of coagulant required, the resulting increase in permanent hardness (as calcium sulphate) is only p. p. m., practically negligible. The combination of softening by use of lime, small amoun of coagulant, and naturally low permanen hardness, give a water that not only meets a domestic requirements, but is excellent fo boiler and general manufacturing purposes.

Boiler trials conducted at Kirtland Pumping Station with water from the test plan show a very marked reduction in scale formation over raw lake water. As 1/16 in. of scal in a boiler may cause a decrease in efficienc of from 5 to 10 per cent, it will be seen th this has an important bearing in the case o a large manufacturing community.

Referring to Fig. 2, the cribs and intak tunnels, Kirtland and Division pumping stations, the booster station, and Fairmont an Kinsman reservoirs shown are existing structures. The 7-ft. tunnel to Division pumpin station is broken and that pumping station i practically abandoned, being used only t boost water from the city mains to the fir high service district. This tunnel is being repaired and will be extended farther into th lake and the pumping station will be practically rebuilt in connection with the filtratio project.

The present handling of the water suppl of Cleveland is as follows: The water i taken from Lake Erie at the 4-mile crib an from thence passes through the new 9-f tunnel to the Kirtland pumping station. At this pumping station 25,000,000-gal. high duty pumps discharge into the distribution systems. The surplus passes to the Fairmont Reservoir. The booster station lifts water from this reservoir to the Kinsman Reservoir which supplies the higher level. A small area above this level is supplied by a second booster pumping to a 100,000-gal tank. Cleveland appears flat, but there is a gradual increase in elevation away from the lake.

It is proposed to build two filtration plants, one at the old Division avenue pumping station, of 150,000,000 gals. per day capacity, and the second, a 75,000,000 gal. plant, near the Kirtland street station. The Division avenue plant will be built first, and will supply the whole city until the second plant can be completed. The Division pumping station will be rehabilitated, and equipped with centrifugal pumps, either engine or turbine driven, for raising the water from the intake tunnel to the filter plant, and equipped with high service pumps to deliver filtered water to the distribution system and reservoirs. A new tunnel extending beyond crib No. 4 is being built to supply this filter plant. Should the plant be completed prior to the tunnel, raw water will be pumped to it from Kirtland station, and repumped to the distributing system after filtration.

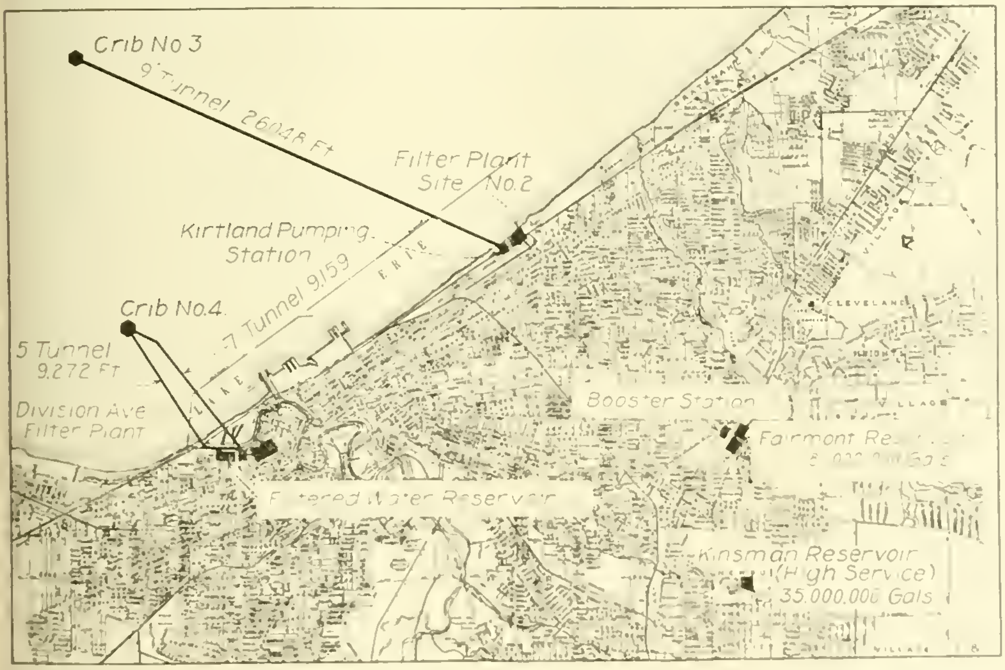


Fig. 2. Map of Portion of Cleveland, Ohio, Showing Present Intake Cribs, Pumping Stations and Distribution Reservoirs, and Location of the Two Proposed Water Filtration Plants.

The Division Avenue Filtration Plant is now under construction. Intake Cribs Nos 1 and 2 are no longer in existence.

The Division avenue filter plant will comprise a filter and administration building, mixing and settling basins, chemical house and filtered water reservoir. The administration building will occupy a central position and will contain the offices and laboratories. It is intended to locate all gages, sampling pumps, meter dials, etc., in this building, making it as far as possible the nerve center for the entire plant. There will be 36 filter units, each of 1,166,660 gals. capacity. These will be arranged along two galleries, with nine filters on either side of each gallery, extending from the east and west ends of the administration building and connecting directly thereto.

The filter equipment and piping will be of the usual type and the filtering material will consist of 30 ins. of sand supported by 12 ins. of graded gravel. Wash water will be supplied from a reinforced concrete reservoir located on the bluff above the plant. Before entering the sedimentation basins proper the chemically treated water will pass through a mixing basin. This is very closely baffled, so that the chemicals in the water will be given every opportunity to react with the hardening constituents present. This will greatly accelerate the naturally slow interaction between the calcium hydroxid and the actual bicarbonates present in the water. It will require 30 minutes for the water to pass through the mixing basin. There will be six sedimentation basins, each of 3,650,000 gals. capacity, in which the sediment and chemical precipitates will settle out before the water passes to the filters. This gives 3 hours' sedimentation at full capacity of plant.

TABLE 1.—AVERAGE ANALYSIS OF LAKE ERIE WATER, CLEVELAND, O.

	Symbol.	P.P.M.
Calcium carbonate	$CaCO_3$	82.0
Magnesium carbonate	$MgCO_3$	15.0
Magnesium sulphate	$MgSO_4$	16.0
Magnesium chlorid	$MgCl_2$	2.0
Iron and aluminum (probably colloidal)	Fe & Al	0.5
Silicon (probably colloidal)	Si	1.0
Sodium chlorid	NaCl	22.0
Free carbonic acid	CO_2	5.0
Half-Bound carbonic acid	HCO_3	44.0
Hardness (total)	......	110
Alkalinity (total to Erythrosine)	......	100

The chemical house will be designed to handle lime and coagulant received in bulk from cars on an adjacent siding. The chemicals will be raised by conveyors to air tight storage bins, whence they will be fed by gravity through automatic measuring devices into solution tanks and discharged by centrifugal pumps into the water as it enters the mixing basins.

A filtered water reservoir of 25,000,000 gals capacity will be built near the plant and will furnish ample reserve for all fluctuations.

Work on this plant is being pushed with all possible speed. The excavation was started Jan. 27, 1914, and is being done by the Water Department. Two steam shovels are being used, the excavated material being hauled to the easterly part of town by the Cleveland Railway Company and used for filling in low ground. Construction will be divided into five contracts: No. 1, concrete work on filters, sedimentation and mixing basins, and main piping; No. 2, chemical house; No. 3, filter superstructure; No. 4, filter equipment; No. 5, filtered water reservoir. Plans for contract No. 1 will be ready for bids early this spring. The remaining contracts will follow in order.

This work is being done under the Department of Public Utilities, Mr. C. W. Stage, director; Mr. C. F. Schulz, water commissioner, and Mr. C. P. Jaeger, deputy commissioner. Mr. R. W. Pratt is consulting engineer and is in charge of the filtration work. Mr. Jaeger has personal charge of the new tunnel and intake.

Notes on Fire Protection Requirements of the Water Works in a Town of 4,000 Population.

The present article discusses, from the standpoint of the fire-protection engineer, the fire protection requirements of the water works in a town of 4,000 population. The information given is taken from a paper on Fire Protection and Its Relation to the Fire Waste, which was read before the Nashville section of the Engineering Association of the South, by Mr. H. B. Long, and published in the first quarterly journal of the Association for 1914. The principal requirements for grading a town of 4,000 population as 4½ class are outlined. Mr. Long states that before submitting water works plans and specifications to a municipality engineers should freely consult with the underwriting authorities to the end that such a water works system be installed as will grant the citizens the lowest possible insurance rates for the protection afforded.

Classification of Towns.—All fire insurance rates are based on what is known as "town classification," and towns are classified by the efficiency of their fire-fighting facilities. Six classes of towns are recognized. A sixth class town has no fire protection, while a first class town has a high-pressure fire protection system and the other components of a perfect fire-fighting equipment. A fifth class town is one with a chemical engine or a very deficient water works or fire department equipment. About 75 per cent of the Tennessee towns having water works systems measure up to the 4½ class, and designing engineers are most frequently called upon to design systems of this size.

Number of Fire Streams.—In determining upon the number of fire streams required each town will require specific treatment, the number of standard 250-gal. fire streams being determined according to the building construction and the liability to a general conflagration. The building construction of one town of 4,000 population may be of such a quick-burning construction as to require ten or more good fire streams to prevent a general conflagration, while in another town of the same population the building construction may be of such fire-resisting material that three streams would likely be sufficient. However, experience has shown that the average town of 4,000 population that is graded as 4½ class requires four standard fire streams, or a total discharge of 1,000 gals. of water per minute. This amount, added to the domestic consumption of 100 gals. per capita would call for a demand of approximately 1,278 gals. of water per minute, and upon these figures storage and pumping capacities must be based.

System of Operation.—It is the practice of some towns to provide a comparatively large elevated storage supply and small pumping equipment, while others take the opposite view and provide a small elevated tank and a comparatively large pumping equipment. When in good condition, either system will furnish the required fire flow, and would be acceptable as far as classification is concerned. The combined system, having a large elevated storage reservoir, closely approaches the ideal features of the gravity system, and furnishes a constant, unvarying pressure that is preferable to direct pumping.

For reliable operation the plant that is operated by steam-driven pumps requires more than one boiler, in order that the service may not be impaired while repairs are being made; likewise more than one pump is required, in order that the equipment will be in working order at all times.

Pumps.—The National Fire Prevention Association has drawn up specifications of centrifugal, rotary, and steam-driven duplex pumps that have proved reliable, and are recommended for fire service. The principal points of difference between the national standard steam duplex pump and the ordinary "trade" fire pump are as follows:

1. Its steam ports and water passages, also air chambers, are made much larger than in common trade pumps, so that a larger volume of water can be delivered in an emergency without water hammer.

2. It is "rust-proofed" that it may start instantly, after disuse, by making its piston rods and valve rods of Tobin bronze instead of steel, and its water pistons, stuffing boxes, and shaft bearings of brass instead of cast iron. Its valve levers are made of steel or wrought-iron forgings or of steel castings.

3. The following necessary attachments are included in the price of the national standard pump: A vacuum chamber, two pressure gages, a relief valve, a set of brass priming pipes, two to six hose valves, a stroke gage, a capacity plate, an oil pump, a sight-feed lubricator, and a cast iron relief-valve discharge cone.

By reason of the larger ports, passageways and pipes, its larger number of valves, and the added attachments and general stronger and superior construction, the national standard pump costs more than a common trade fire pump; but the cost per gallon which these pumps can deliver in an emergency by reason of their large passageways, etc., is no greater than that of the old style of fire pump, and is well worth the extra cost.

Distribution System.—The best arrangement of pipes for the distribution of water over a given territory is by the "gridiron" system. From the standpoint of economy as well as efficiency large feeders that will transmit the amount of water required with low velocities are preferable; and in designing new systems, no pipe smaller than 6 ins. should be used, even in the residential section of a town. Perhaps in many cases 4-in. pipe will adequately serve a town's present needs in the residential districts for connecting up dead ends, etc.; but in order to provide for the town's growing needs, a generous amount of 6-in. and larger size mains should be used. This will then allow for considerable extensions to be made that will transmit the required amount of water at low velocity.

Gate Valves.—The question of gate valves should be given the designing engineer's close attention. Of course the ideal condition would be to have gate valves located at all cross-connecting lines in order that not more than one block of mains would be put out of service in case of a break, but in complying with 4½-class requirements, valves that are placed every 600 ft. in the congested value district and not more than 1,000 ft. apart in the residential district would be acceptable.

Hydrants.—The question of the type of hydrants is left for the designing engineer to decide for himself. The only suggestions are that all hydrants be of frost-proof design and have not less than 6-in. barrels and two 2½-in. discharge outlets. It is strongly recommended that, wherever funds permit, all be provided with individual gate valves and be set in gravel to insure reliable drainage

Hydrant Spacing.—The proper spacing of hydrants is a question that in the past has been given altogether too little attention. A liberal distribution of hydrants is both a matter of economy and convenience, for this will avoid long lengths of hose, with their attendant loss of pressure head. Furthermore, hose lasts from 5 to 10 years; whereas the life of a good hydrant is 50 years, and a hydrant costs only one-half as much as one length of good hose. Considering the increased efficiency and saving in wear and tear on fire hose, it is of the greatest economy to install more hydrants and use less hose.

Friction in Fire Hose.—A very unfortunate condition that exists in the installation of a new fire-protection equipment lies in the fact that usually an amount of money in excess of a town's appropriation is spent in installing the water works end of the protection, and funds will not always permit the purchase of fire-department equipment of the best grade. It is false economy to purchase other than first-quality hose, for fire-department hose of an inferior quality is largely made of reclaimed rubber and a poor grade of fabric, which, when placed in service, shows an excessive loss in pressure, due to a rough rubber lining and a great amount of elongation and twisting, which is very detrimental, especially when used in localities where the pressure is poor. The loss of pressure due to hose friction varies greatly with the different brands of hose. Mr. Long has made many tests of different brands of 2½-in. rubber-lined hose, and has found the friction loss to range from 11 to 20 lbs. in pressure for 100 ft. of hose when discharging 250 gals. per minute. It is very evident from these figures that the most efficient fire fighting can be accomplished by the use of a good quality of hose that will conduct the greatest amount of water to the seat of a fire with the least friction loss, as well as economy of operation when direct pumping systems are used.

Size of Nozzles.—Effective fire fighting is only accomplished by the use of nozzles of sufficient size that will throw a solid body of water into a fire that will leave a black mark wherever it strikes. The size of nozzles to be used will depend entirely upon the area of the fire and the intensity of the heat. In small-dwelling fires ¾-in. and 1-in. nozzles are usually used, while in fires of a greater magnitude nozzles as large as 2 ins. are required to lower the temperature of burning substances below their ignition points. Fire-protection engineers have agreed that the 1⅛-in. tip is the most desirable for fighting the average fire; and when a pressure of 45 lbs. is provided at the nozzle, the 1⅛-in. tip discharges 250 gals. per minute, which is designated as a standard fire stream.

Loading and Unloading Problem Solved.—An interesting problem in hauling heavy materials from freight cars to the job was solved lately by J. F. Culp & Sons, Buffalo, N. Y., who own four Pierce-Arrow five-ton trucks. This firm had the contract for hauling all the marble for the new Catholic cathedral, Delaware avenue and Utica street, of that city. The total amount of marble used in the building is a little over 590 carloads—450 cars of exterior and 140 cars of interior. The total cost of the building is nearly $1,-500,000.

Some difficulty was experienced at first in the handling of the marble both at the car end and at the building site. It was found that a great deal of time was used up in loading. This had to be done by a crane, while the unloading was all done by hand, piece by piece. The company soon conceived the idea of using skips or shallow boxes. These boxes were filled at the cars while the truck was making a trip, and on its return loaded in a few seconds by the crane. At the building a platform the height of the truck was built with a sloping runway to the ground. The boxes were fitted with small wheels and could be easily pushed from the truck to the platform, and from there to the ground.

Breakage was practically nil, and both contractors and engineers state that they were never held up for stone on account of the trucks.

One truck in a nine-hour day hauled from fifty to fifty-five tons of marble, the round trip averaging about eight miles.

Bridges

Design and Construction Features of the Milwaukee Ave. and Desplaines St. Viaducts, Chicago.

Contributed by J. H. Prior, Assistant Chief Engineer, Illinois Public Utilities Commission, formerly Engineer of Design, Chicago, Milwaukee & St. Paul Ry.

Milwaukee Ave. is one of the fourteen farm roads which radiated from the early village of Chicago. At that time it was one of the principal highways, and it has retained its importance as a highway, as approximately 30,000 people pass over it daily in and out of the business district of Chicago.

The viaducts described in the following article carry Milwaukee Ave. and Desplaines St. over the tracks and team yards of the Chicago, Milwaukee & St. Paul, the Chicago & Northwestern, and the Pennsylvania railroad companies, between Kinzie St. and Wayman St. At this location the tracks of these companies are just approaching the central business district of Chicago, occupying a belt 460 ft. wide, as shown in Fig. 1. Milwaukee Ave. crosses these tracks in a diagonal direction, intersecting Desplaines St. on the north in such a manner as to form a V-shape figure, one arm being formed by Milwaukee Ave. and the other by Desplaines St., the north ends of the viaducts being common to both streets. The Desplaines St. viaduct crosses the tracks at an angle of approximately 90°, and it is 521 ft. long. The total length of the Milwaukee Ave. viaduct, which crosses the tracks diagonally, is 665 ft.

The old viaducts on both Milwaukee Ave. and Desplaines St. were built in 1873. There

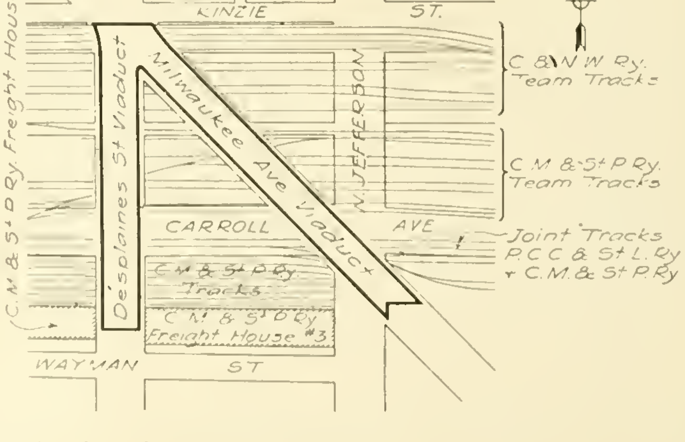


Fig. 1. Map Showing Location of Milwaukee Ave. and Desplaines St. Viaducts, Chicago.

is no record of the specifications and loading for which they were designed, but durng the last few years of their life various parts of the structures were overstressed more than 100 per cent under traffic. This ondition was remedied by the decision to relace these bridges with new structures deigned under the city specifications, the loadng for which is substantially equivalent to ooper's "Class A-2" loading for highway ridges. The new structures are both heavier han the original ones, and have the widths of roadways and sidewalks shown in Figs and 9.

DESIGN FEATURES.

Preliminary estimates showed a reinforced oncrete structure to be cheaper than a steel tructure with a concrete floor, besides requiring less expense for maintenance. It was therefore decided to build eight reinforced concrete spans on the north end of each structure, with a truss and a plate girder span on the south end of the Milwaukee Ave. viaduct and three plate girder spans on the south end of the Desplaines St. viaduct, the latter requiring longer spans than could be made of concrete. As the truss span and plate girder spans on the south end of the Milwaukee Ave. viaduct (see Fig. 3) and the girder spans on the Desplaines St. viaduct present no unusual features, this article will consist mostly of a description of the eight reinforced concrete spans on the north end of these viaducts.

In general, the concrete was a 1:2:4 mix, crushed limestone being used, but that used for the columns was a 1:1 mix, and for some of the footings a 1:3:6 mix was used. Tests of the concrete at 28 days showed a compressive strength for cylinders 8 ins. in diameter by 16 ins. long of 2,500 lbs. per square inch for 1:1 concrete, and of 2,000 lbs. per square inch for 1:2:4 concrete. The cement gun was used for pointing after the forms were removed.

In designing the reinforced concrete spans it was necessary to meet three rather unusual requirements. The first of these requirements was that the new bridges should carry the heaviest city traffic, with a floor depth not much greater than that used in the old steel structure, which was designed for the light traffic of 40 years ago. It was not difficult to meet this requirement in the design of the Desplaines St. viaduct, as the city consented to a maximum rise of 8½ ins. in the street grade. For the Milwaukee Ave. viaduct, however, it was necessary to design through girder spans, as the street grade established by the city, together with the headroom which had to be provided over the railway tracks below, limited the floor depth to 1 ft. 1 in., from top of street car rail to bottom of concrete. The outlines of a typical span are shown in Fig. 2. The main through girders "G" have an average length of 55 ft. and a maximum length of 57 ft. 1 in. These are the longest railroad bridge girders of reinforced concrete known to the writer to have been constructed.

As the main girders are a little more than 42 ft. apart, and as the average perpendicular distance between the bents is slightly less than this, the beams "b_3" were placed at right angles to the bents "C" instead of at right angles to the girders "G," as is more usual in through girder construction.

The beams "b_3" afford nearly all the support for the heavily loaded central portion of the roadway under the street railway tracks. The beams "b_1" and "b_2" are also at right angles to the bent "C," and carry most of the more lightly loaded portion of the roadway. One end of these beams is supported by the bents "C," and the other end is supported by the main girders "G."

This arrangement of the floor system has the additional advantage of reducing the total load to be carried by the girder "G," thus avoiding the inconveniently large dimensions which would otherwise exist. By arranging the beams in this manner, and by the use of a special rail fastening, described in another part of this article, it was possible to con-

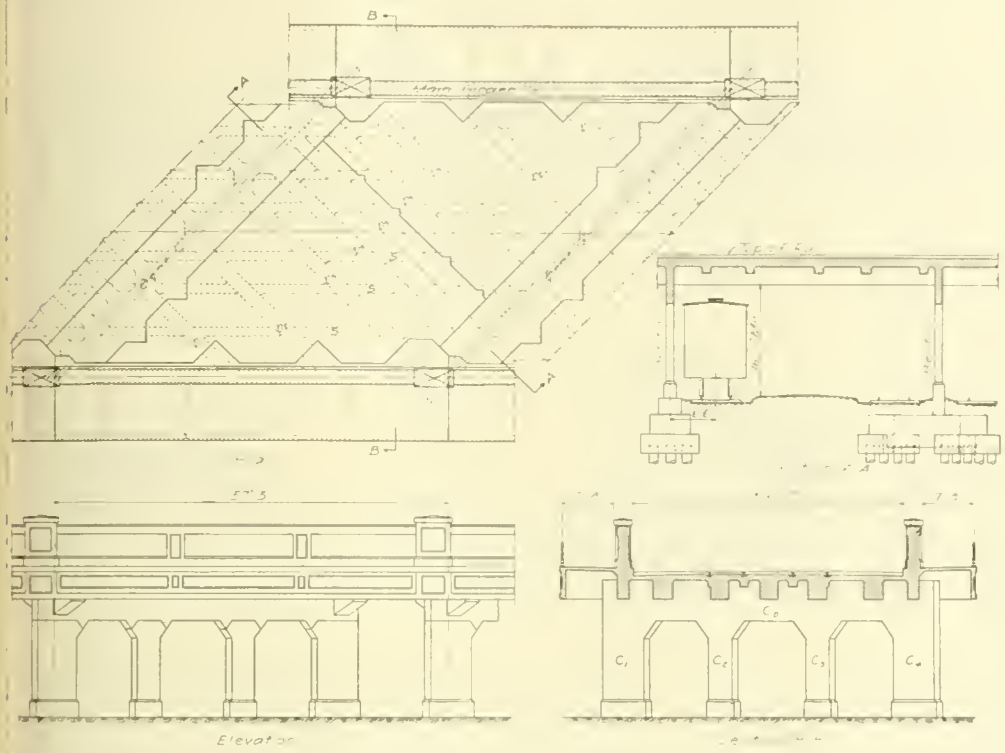


Fig. 2. Plan, Elevations and Cross-Sections of Typical Panels of Milwaukee Ave. Viaduct.

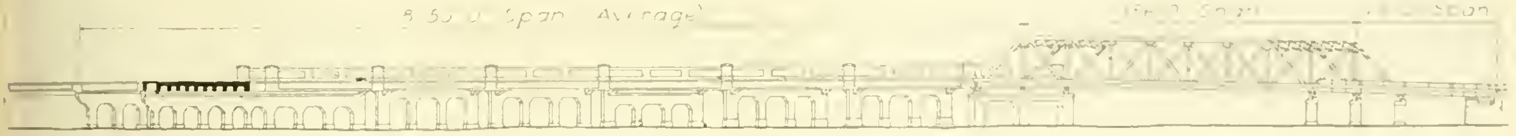


Fig. 3. Elevation of Milwaukee Ave. Viaduct Showing Type of Construction and Span Lengths.

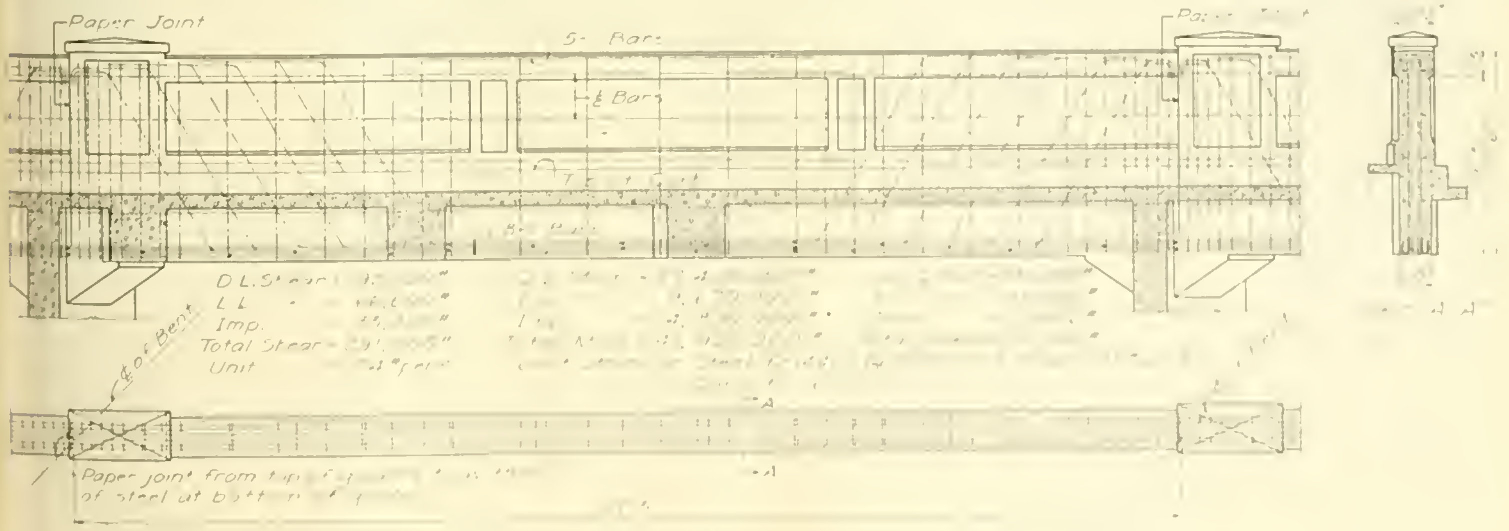


Fig. 4. Data for Designing and Details of Main Girders—Milwaukee Ave. Viaduct.

struct the floor with the required depth of 4 ft. 1 in. In the opinion of the writer, the design would have been complete without the short stringers "S" (Fig. 2), which extend between the beams "b_2," "b_3," etc., in a direction parallel to the center line of the viaduct. However, these stringers were inserted at the direction of the municipal authorities.

The general details of the main girders of the Milwaukee Ave. viaduct are shown in Fig. 4. These girders are 10 ft. 6 ins. deep and 20 ft. wide, and are reinforced with twenty-eight 1-in. bars spaced 3 ins. horizontally and 2 ins. vertically. They also contain a very considerable amount of vertical reinforcement, which was necessary to provide for web stresses.

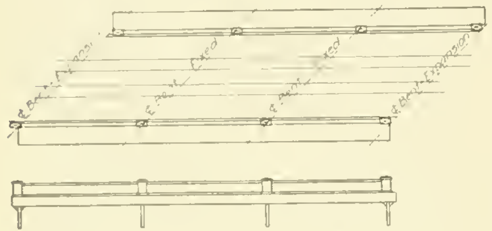


Fig. 5. Typical Group of Three Spans—Milwaukee Ave. Viaduct.

The floor construction of the Desplaines St. viaduct is similar to that of Milwaukee Ave. viaduct, except that there is no occasion for heavy girders, as the bridge crosses the tracks at approximately 90°.

The concrete spans in both viaducts are arranged in groups of three, as shown in Fig. 5. The ends of the span which rest on the two center bents of each group were made fixed ends, and the ends which rest on the outside bents were made expansion ends. The fixed ends of the beams, "b_1," "b_2" and "b_3" (see Fig. 2) were built as a monolithic with the cross beam "C_0," but at the expansion end they rest upon the bent "C_2," and are tied together by a diaphragm, expansion being provided for by means of a specially designed sliding bearing.

In the Milwaukee Ave. viaduct, the main girders "G," at their expansion end, are carried on the expansion bearings shown in Fig. 6. The lower half of these bearings consist of two beam grillages, "g_1" and "g_2," concreted into the bent. The lower beams, "g_2," of this grillage were made parallel to the bents to assist in distributing the load over the bent, while the upper beams "g_1" were made parallel to the center line of the girders to afford a sufficient bearing area along the bottom of the girders. Bronze plates, "A" and "B," the upper surfaces of which were finished as a sliding surface, are fitted into the upper cover plates "E" of the grillage beams "g_1" by means of a lug. Cast steel plates, "C" and "D," planed smooth and designed to slide upon the bronze plates "A" and "B," were concreted into the under surface of the main girder "G." It is believed that this detail will be a protection against excessive stresses due to the expansion and contraction of the concrete structure, and that it will also prevent the formation of cracks around the bearings which are sometimes found in concrete structures not provided with adequate expansion joints. A similar provision was made for expansion under each of the beams "b_1," "b_2," etc., where they rest on the bents.

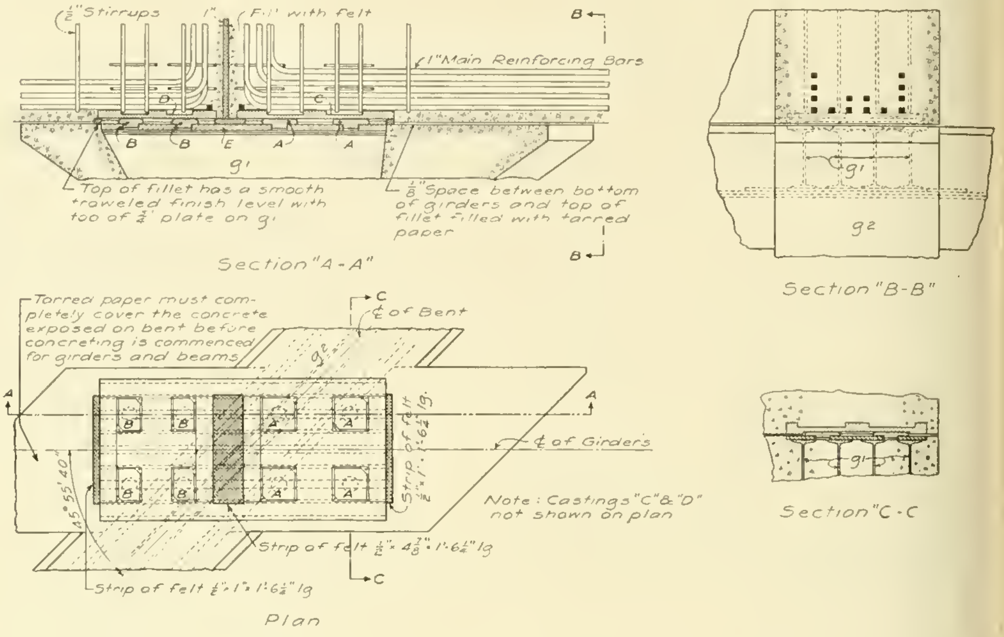

½" Stirrups
1"
Fill with felt
B
1" Main Reinforcing Bars
D
C
B
B
E
A
A
g1
Top of fillet has a smooth traweled finish level with top of ¾" plate on g1
⅛" Space between bottom of girders and top of fillet filled with tarred paper
B
Section "A-A"
g1
g2
Section "B-B"
Tarred paper must completely cover the concrete exposed on bent before concreting is commenced for girders and beams
C
℄ of Bent
g2
A
A
B
B
A
A
B
B
A
A
℄ of Girders
Strip of felt ½" × 1" × 1'-6¼" lg
45° 55' 40"
Note: Castings "C" & "D" not shown on plan
Strip of felt ½" × 4⅞" × 1'-6¼" lg
Strip of felt ½" × 1" × 1'-6¼" lg
C
Plan
g1
Section "C-C"

Fig. 6. Details of Expansion Bearings for Main Girders of Milwaukee Ave. Viaduct.

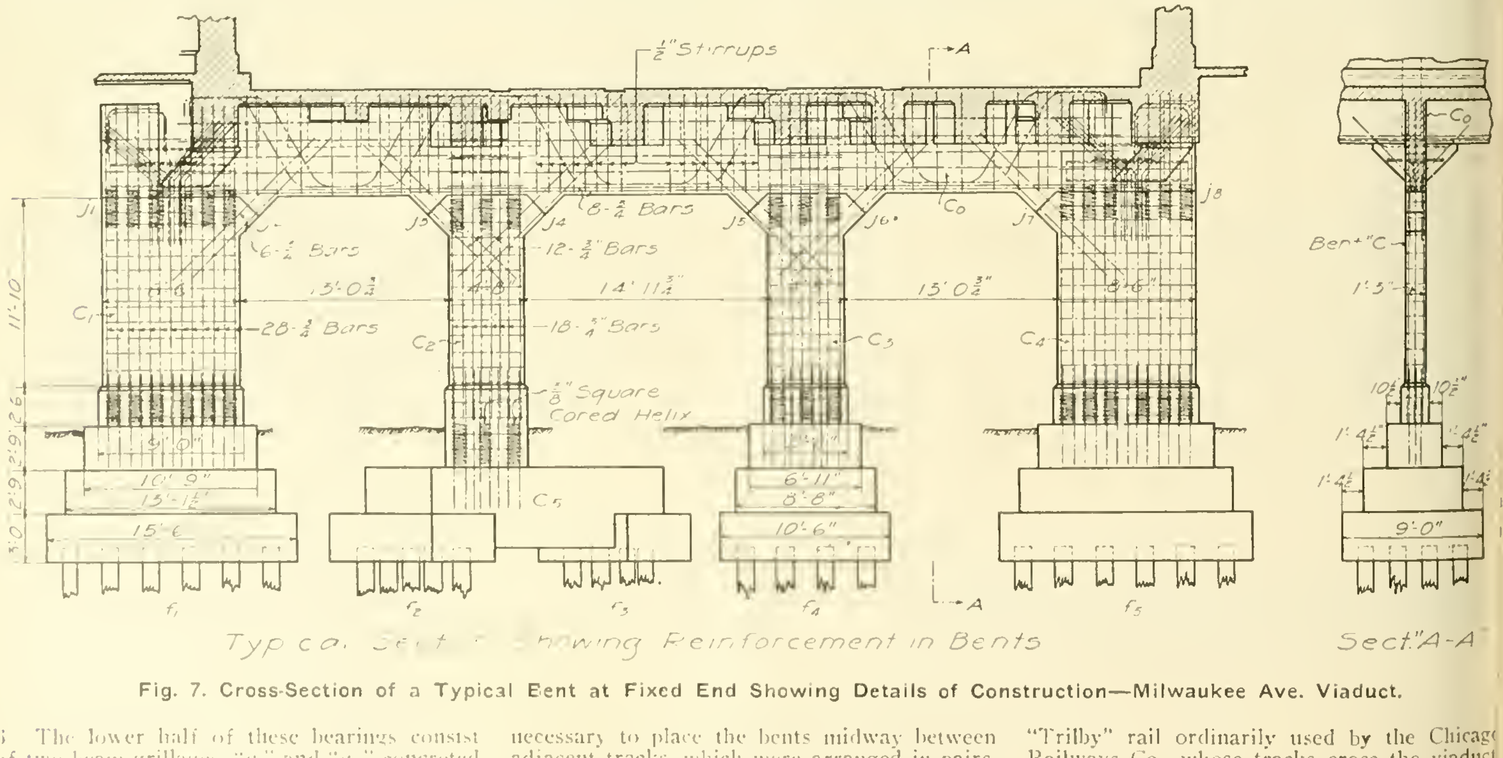


Fig. 7. Cross-Section of a Typical Bent at Fixed End Showing Details of Construction—Milwaukee Ave. Viaduct.

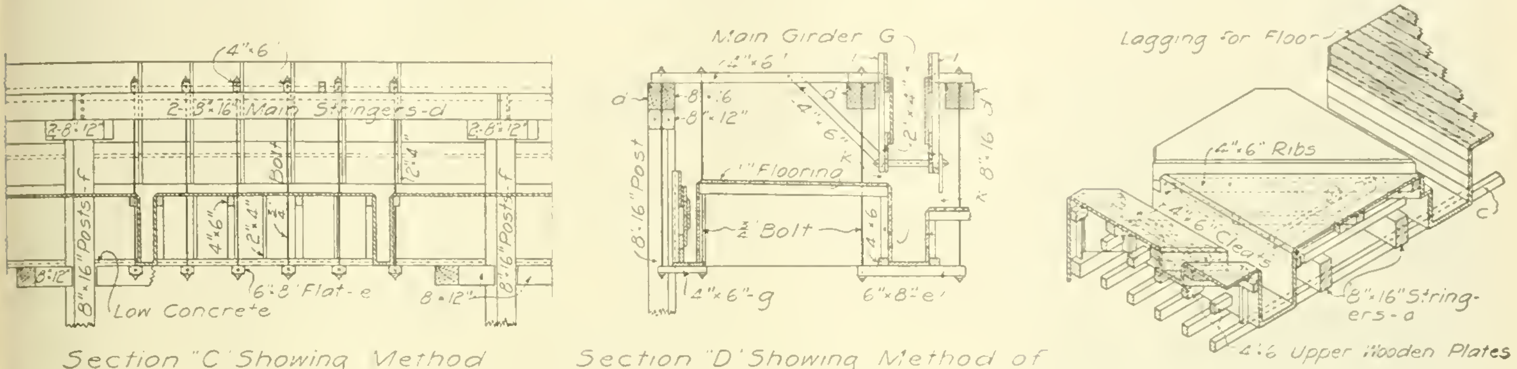
The details of a typical bent are shown in Fig. 2 and Fig. 7. In order to maintain the former clearances for teaming purposes it was necessary to place the bents midway between adjacent tracks, which were arranged in pairs, as shown in Fig. 8. (This drawing also shows the typical arrangement of forms and form supports). This limited the thickness of the bents to 1 ft. 3 ins., necessitating the use in the columns of 1 per cent of vertical reinforcement, with hoops and spirals. The column "C_1" (Fig. 7) is supported directly upon a pile foundation, but the column "C_2" is supported upon the beam "C_5" which spans the tunnel of the Illinois Tunnel Co. This tunnel runs directly beneath both viaducts, and is about 50 ft. under ground. The ends of the beam "C_5" are carried upon the pile foundations "f_2" and "f_3," while the columns "C_1," "C_2," etc., support the cross beams "C_0."

The bent shown in Fig. 7 is typical of the bents which carry the fixed end of the girders. A construction joint was made between the columns and the beam "C_0," which was built as a monolith with the columns.

Figures 9 and 14 show the details of a bent similar to the one last described, but located on the Desplaines St. viaduct.

With the depth of floor and the long spans required on Milwaukee Ave. it was not possible to use ties and ballast with the 9-in "Trilby" rail ordinarily used by the Chicag Railways Co., whose tracks cross the viaduct A special rail chair was therefore provide for a 7-in. "Trilby" rail, which the Chicag Railways Co. consented to use. This reduce the available floor depth from 4 ft. 1 in. t 3 ft. 6 ins., whereas, if ties and ballast ha been used, the available depth would have bee reduced an additional 7½ ins., besides greatly increasing the dead load of the floor.

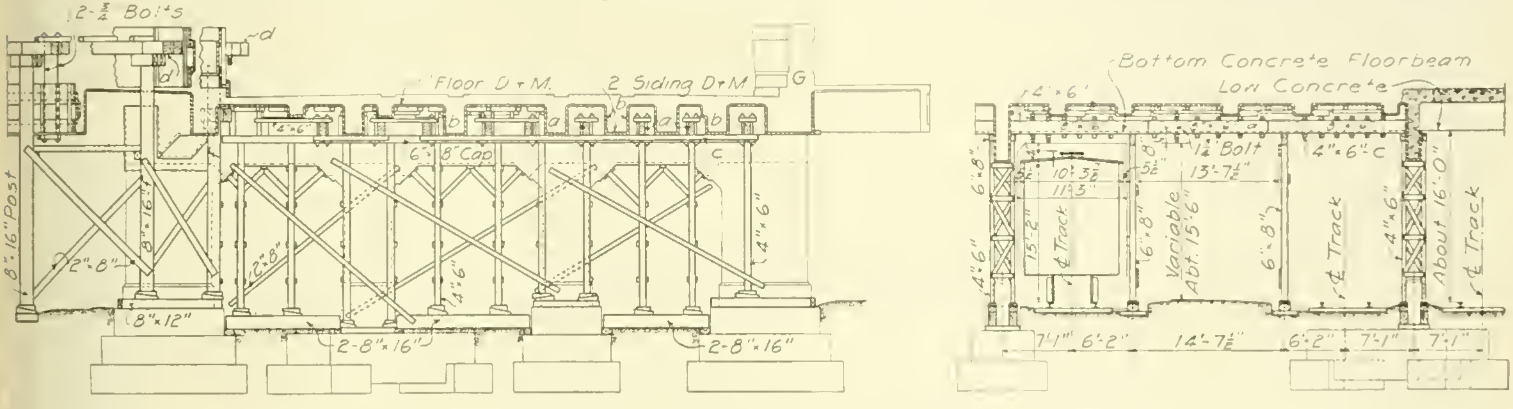
The rail chair which is shown in Figs. 10 and 11 was designed so as to interfere as little as possible with the beam action of the concrete and with the reinforcement of the slabs when placed in the structure. It was also intended to produce a suitable anchor and sufficient bearing for the rails. This was accomplished by making the main part of the rail chair in two pieces, the bed plate and the anchor plate, which are entirely independent of each other.



Section "C" Showing Method of Supporting Girders

Section "D" Showing Method of Supporting Forms for Girders, Sidewalks and Curtain Walls

Isometric Section



Section "A" Showing Supports for Forms at Concrete Bents

Section "B" Showing Clearances and Main Supports for all Spans Between Bents No. 10 and Pier No. 3

Fig. 8. Forms and Form Supports for Various Parts of Concrete Spans—Milwaukee Ave. Viaduct.

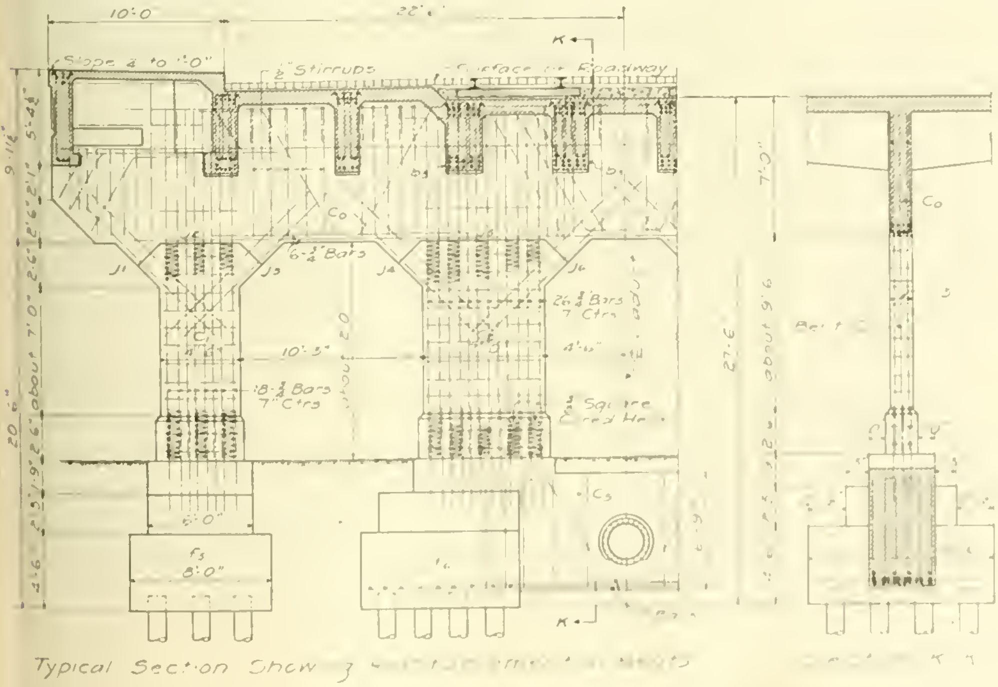


Fig. 9. Cross-Section of a Typical Bent Showing Details of Construction—Desplaines St. Viaduct.

The bed plate "G" (see Fig. 11) is approximately 16 ins. square and is 1½ ins. thick, the perimeter being embedded in the concrete. There are four square holes in the bed plate, through which the sockets of the separate anchor plate "*H*" extend. This anchor plate is a casting, with a lower bar running transversely with the rail. There are two upright projections, "*K*," each of which terminates in a socket for receiving the head of a T-bolt. The anchor plate is placed in the floor between the upper and lower slab reinforcement, thus insuring a good anchorage when entirely surrounded with concrete. The rail is held in position by means of T-bolts and clamps "*F*" and "*E*," the T-bolts extending down through the holes in the bed plate to the sockets in the top of the anchor plate. A 1-in. wearing plate was provided between the bed plate and the rail and screw-bolted to the bed plate. This wearing plate can be renewed by removing the screw bolts, clamps and T-bolts.

In constructing the viaduct the chairs were clamped to the rail at 3-ft. centers, and the rail was aligned and placed at its proper elevation before concreting. The 1-in. wooden washers shown in Fig. 11 were used for the purpose of erection, their object being to act as a temporary filler and to allow the anchor plate to be drawn tightly against the wearing plate. The clamps "*F*" and "*E*" pressed down on the rail base and bed plate, while the heads of the T-bolts pulled up on the anchor plate, thus holding the chair in place until after the concrete was poured. These washers, however, did not prove satisfactory, as the tendency was to tighten the bolts too much, thus causing the wearing plate to bend and a gap to be opened between it and the base plate. This defect was remedied in the field by placing small wooden blocks between the outer ends of the anchor plate and the bed plate and by omitting the wooden washers. On the Desplaines St. viaduct, owing to the increased thickness of floor, it was not necessary to use special rail chairs and ties and ballast were used, as shown in Fig. 9.

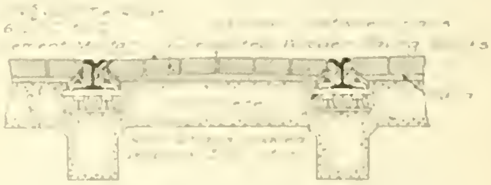


Fig. 10. Cross-Section Through Track Showing Details of Rail Chairs for Milwaukee Ave. Viaduct.

The second condition imposed on the design was that the bridges be constructed with only

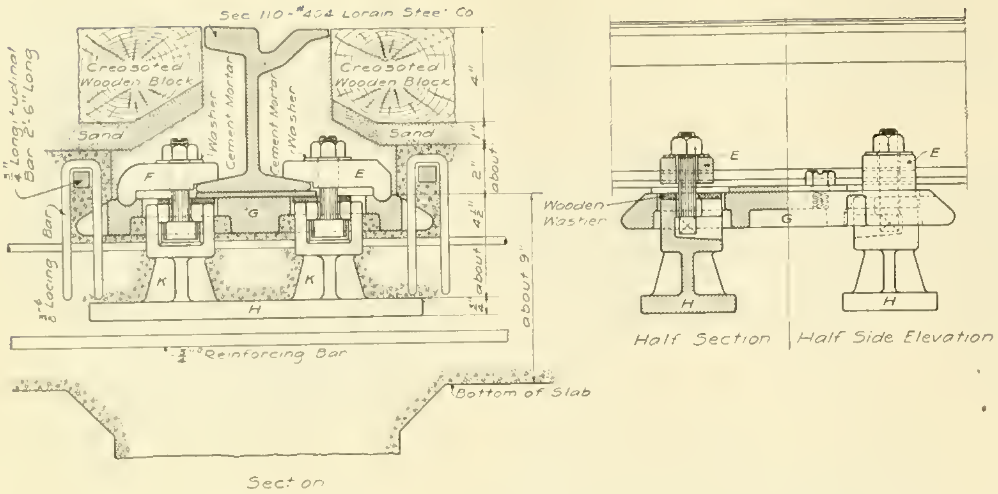


Fig. 11. Detail Drawing of Rail Chairs and Connections for Milwaukee Ave. Viaduct.

a reasonable interference with the properties of the public and municipal utilities at present occupying the surface or sub-surface space. These properties were as follows: The track and wires of the Chicago Railways Co.; the tunnel of the Illinois Tunnel Co.; the lighting and power conduits of the Commonwealth-Edison Co.; the telephone conduits of the Chicago Telephone Co., and the water pipes, sewers and electric light wires and poles of the City of Chicago. Figure 12 shows the location of these properties.

The manner of providing for the tracks of the Chicago Railways Co. has already been described. Their wires were strung overhead from reinforced concrete trolley poles placed over each bent. On Milwaukee Ave. provisions were made for the lighting and power conduits of the Commonwealth-Edison Co. and for the telephone conduits of the Chicago Telephone Co. by embedding 3½-in. "Orangeburg" fiber conduits in concrete in the space directly beneath the sidewalks on each side of the viaduct. On Desplaines St. these conduits were placed in wrought-iron pipes, which were supported on ledges on the stems of the sidewalk T-beams.

The tunnel of the Illinois Tunnel Co. occupies a position in Milwaukee Ave. approximately beneath the center line of the street, at an elevation of about 38 ft. below city datum. It was not possible to secure any support for the viaduct within the space directly above the tunnel. Accordingly, as previously described and as shown in Figs. 7 and 12, the column "C_2," which is located over the tunnel, is supported on a beam "C_5" which spans the tunnel and is supported on the pile foundations "f_2" and "f_3."

The third condition imposed on the design was that the new structure should be built practically without interference with the present railroad tracks and without infringing on their lateral or vertical clearance. This was necessary as the tracks on the lower level were busy terminals of importance to the railroad companies concerned.

To meet this requirement, falsework, designed as shown in Fig. 8, was used on Milwaukee Ave. Section "*B*" shows that the distance from the lowest point of the formwork to the bottom of the permanent concrete floor was only 8 ins. This unusually small projection of the falsework below the concrete was obtained in the following manner: The 8x16-in. stringers "*a*" were placed as shown in Section "*A*," between the stems of the T-beams "*b*" which support the roadway floor. The ends of the stringers "*a*" rest on falsework bents, as shown in Section "*B*." To the underside of stringers "*a*" were bolted 4x6-in. pieces "*c*," which support the bottom and side forms of the T-beams "*b*," as shown in Section "*A*."

FORMS.

The forms for the main girders were supported as follows: 8x16-in. stringers "*d*," (Sections "*A*," "*D*" and "*C*," Fig. 8), rest on the posts "*f*" which pass through temporary openings in the concrete slab. These main stringers "*d*" support the 6x8-in. pieces "*e*" and the 4x6-in. pieces "*g*" by ¾-in. bolts The pieces "*e*" support the bottom and sides of the lower portion of the forms for the main girder "*G*." The forms "*i*," for that par

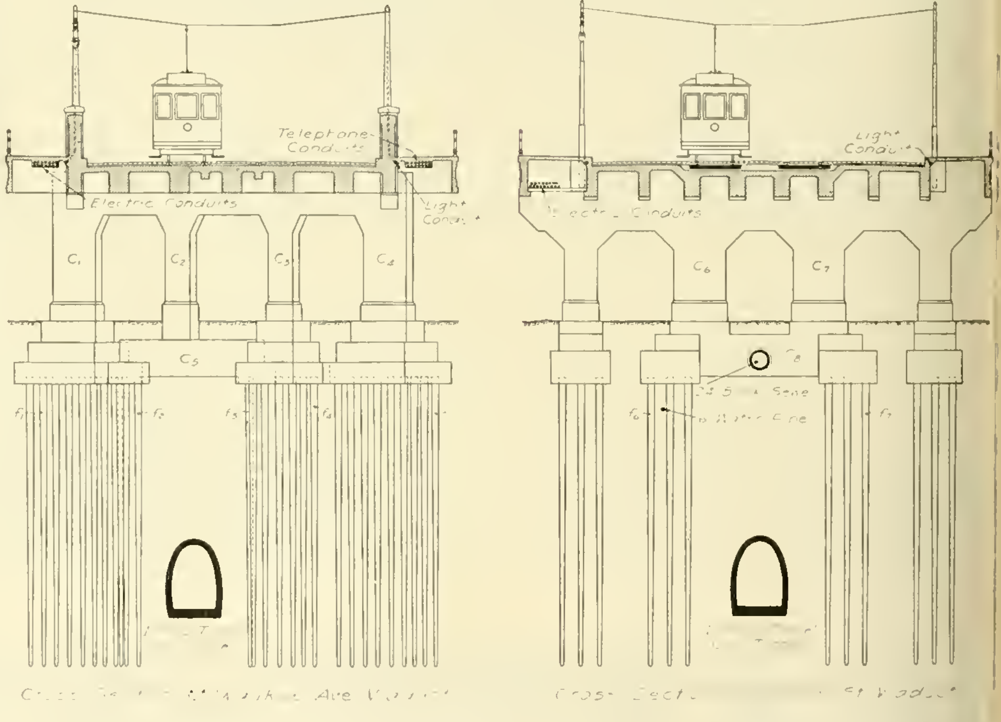


Fig. 12. Cross-Sections Showing Conditions Imposed Upon the Design by Utilities Occupying Surface and Sub-Surface Space—Milwaukee Ave. and Desplaines St. Viaducts.

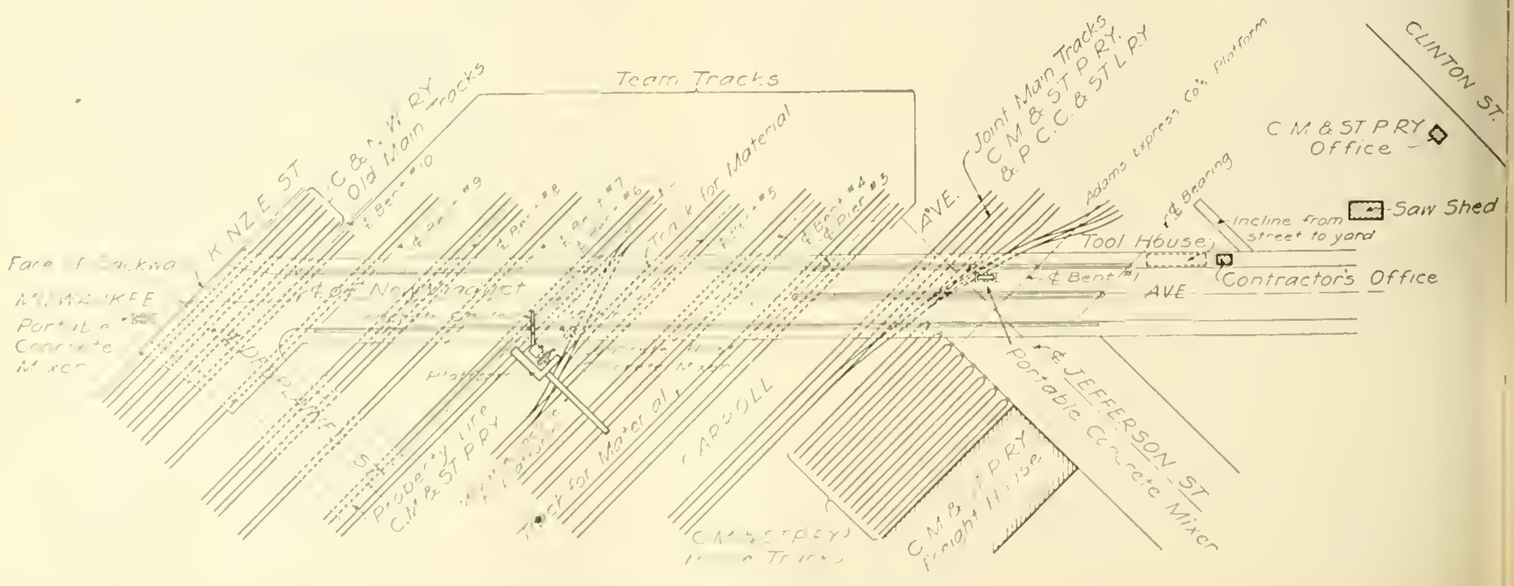


Fig. 13. General Lay-Out Showing Arrangement of Contractor's Equipment for Constructing Milwaukee Ave. Viaduct.

of the main girder "*G*" above the floor slab, are supported directly from the main stringers "*d*" and are independent of the lower portion of the forms "*j*." This arrangement permitted the main girders to be concreted practically as a monolith with the sidewalk and roadway slabs by means of the openings "*k*." The beams "b_1," "b_2," etc. (see Fig. 2) were also concreted as a monolith with the main girders by maintaining similar openings at the ends of those beams.

CONSTRUCTION FEATURES.

In order to interfere as little as possible with traffic crossing the viaducts during their reconstruction, the Milwaukee Ave. viaduct was rebuilt first, the traffic being diverted to he old Desplaines St. viaduct. After the Milwaukee Ave. viaduct was completed, traffic was turned over it, and the Desplaines St. viaduct was rebuilt. Plans for the Milwaukee Ave. viaduct were completed and the structure was 75 per cent completed before plans for the Desplaines St. viaduct were started. The work was done by contract. Separate bids for each viaduct were invited, and the work was done by two different contractors, who used different methods in the placing of the concrete.

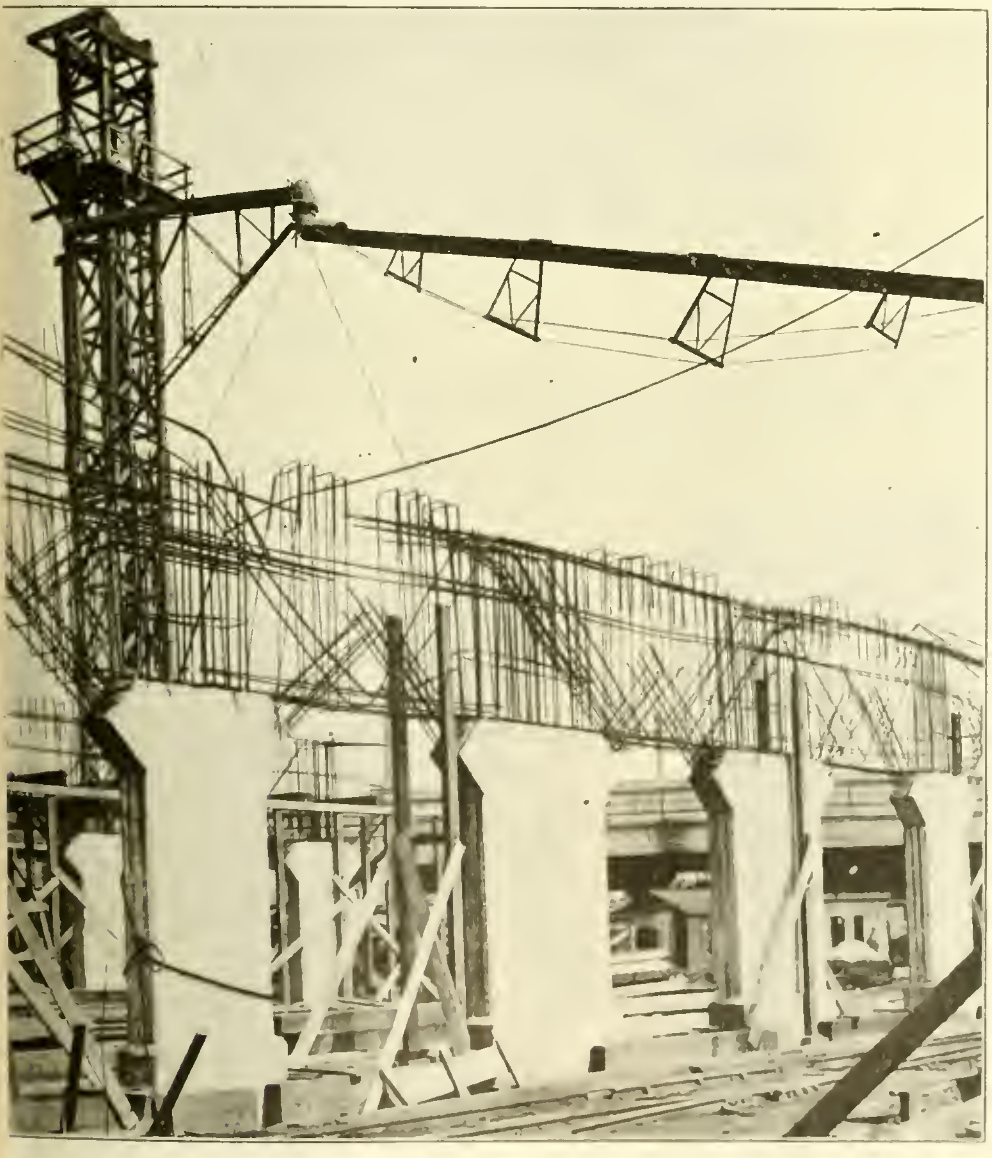


Fig. 14. View of Tower and Chute Used in Construction of Desplaines St. Viaduct—View Also Shows Thin Columns and Reinforcement for Cross Girders—Completed Milwaukee Ave. Viaduct Shown in Background.

Certain conditions were imposed upon the contractors, the main one being that there would be no interference with the operation f trains on the joint main tracks of the Chicago, Milwaukee & St. Paul and the Pennsylvania railroad companies, on the old main tracks of the Chicago & Northwestern company, on the tracks serving the Adams Express Co.'s platform, or on the tracks serving the Chicago, Milwaukee & St. Paul company's freight house. All other tracks under the viaducts are used as team tracks, the cars being spotted at night. These tracks were therefore permitted to be blocked between the hours of 7 a. m. and 6 p. m., but they had to be left open for switching at night. An overhead clearance of 15 ft. 6 ins. above top of rail was required.

In the construction of the Milwaukee Ave viaduct the concrete in the footings, with the exception of the bent between the truss span and the three girder spans, was placed by means of a mixer mounted on a flat car. The car was placed on the tracks adjacent to the footings, and the concrete was deposited through a chute. The materials were taken from the cars and were delivered to the mixer in wheelbarrows. Figure 13 shows the general arrangement of the equipment used by the contractor in placing the concrete in the superstructure. A tower, about 50 ft. high, was erected near the material tracks between bents Nos. 5 and 6. A ½-cu. yd. cube mixer was located on the ground adjacent to the tower, discharging directly into an elevator or bucket. The concrete was hoisted to the top of the tower, dumped into a chute, and distributed by means of push carts of 6 cu. ft. capacity. A large part of the material was delivered to the mixer in wheelbarrows, from cars located on tracks marked "Track for Material," but a portion was brought to the vicinity of the mixer by teams, and was then taken to the mixer in wheelbarrows.

For work at the north abutment and at the two north spans a mixer was placed on the north approach, the concrete being discharged directly into place or distributed in push carts. In concreting the sidewalks and curbs on the steel spans a ¼-cu. yd. mixer mounted on wheels was used, the concrete being distributed in wheelbarrows. Materials were delivered to these mixers in wheelbarrows, from storage piles in the street at each end of the viaduct.

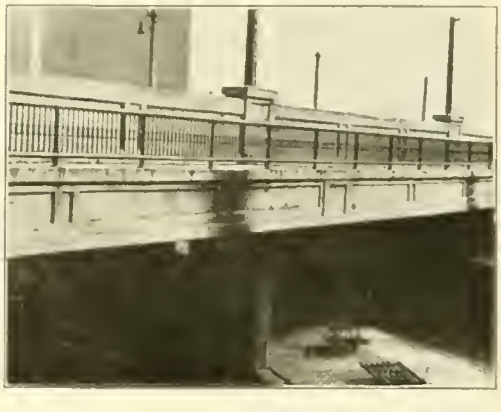


Fig. 15. Portion of Completed Milwaukee Ave. Viaduct Showing Type of Construction.

A portable saw mill was erected near the south approach. Electric power was used in operating the mixers, the elevator, and the saw mill.

In the construction of Desplaines St. viaduct the concrete for both the substructure and superstructure was placed by means of a tower and chute, as shown in Fig. 11, the chute being long enough to deliver concrete to any portion of the structure. Figure 14 also shows the thin columns, the reinforcing in the cross-girders, and the completed Milwaukee Ave. viaduct in the background. Figure 15 shows a side view of this structure after it had been opened to traffic.

In the Milwaukee Ave. viaduct there are approximately 4,000 cu. yds. of concrete and 600,000 lbs. of reinforcing steel, while in the Desplaines St. viaduct there are approximately 2,100 cu. yds. of concrete, and 315,000 lbs. of reinforcing steel.

PERSONNEL

By agreement with the other railway companies involved, the work was designed by the Chicago, Milwaukee & St. Paul Ry. Co., Mr C. F. Loweth, chief engineer, and Mr. J. H. Prior, engineer of design. The concrete work for the Milwaukee Ave. viaduct was built by L. K. Sherman & Co., Chicago, and that for the Desplaines St. viaduct by the Brennen Construction Co., Chicago. The steelwork on both viaducts was erected by the railway company's forces. The field engineering, superintendence and inspection for the railway company, of both the steel and concrete work, were under the direction of Mr. R. J. Middleton, engineer of track elevation for the Chicago, Milwaukee & St Paul Ry Co.

Locomotive Laboratory Tests at University of Illinois.—The University of Illinois has just concluded the series of tests on the Illinois Central consolidation type freight locomotive No. 958, which has been in progress at the locomotive testing laboratory. This engine was furnished by the Illinois Central Railroad for the purpose of these tests and was placed in the laboratory about a year ago. Since that time many tests have been made by both the faculty of the College of Engineering and students in the railway courses. The highest speed at which any tests were made was 15 miles per hour. The greatest tractive effort registered was 30,000 lbs., while the highest horsepower developed was 1,650. The tests were conducted primarily to increase the efficiency of the locomotive performance. Considerable work was also done to improve the running of the locomotive.

Power Plants

Construction and Cost of the Water Tunnel for the Tallulah Falls Hydro-Electric Development in Georgia.

Contributed by Chas. G. Adsit, Engineer Northern Contracting Co., and Eugene Lauchli, Hydraulic Engineer.

The purpose of this tunnel 6,665 ft. long is to convey some 1,700 sec. ft. of water from the storage reservoir formed by a diverting rubble masonry dam 115 ft. high and 440 ft. long, thrown across the Tallulah River, at Tallulah Falls, Ga., to a forebay or surge reservoir, 30 ft. wide, 71 ft. long and 93 ft. high, located at the upper end of six 5-ft. diameter steel penstocks, 1,250 ft. long each, leading to six 17,000-HP. Francis turbines operating under an average head of 580 ft.

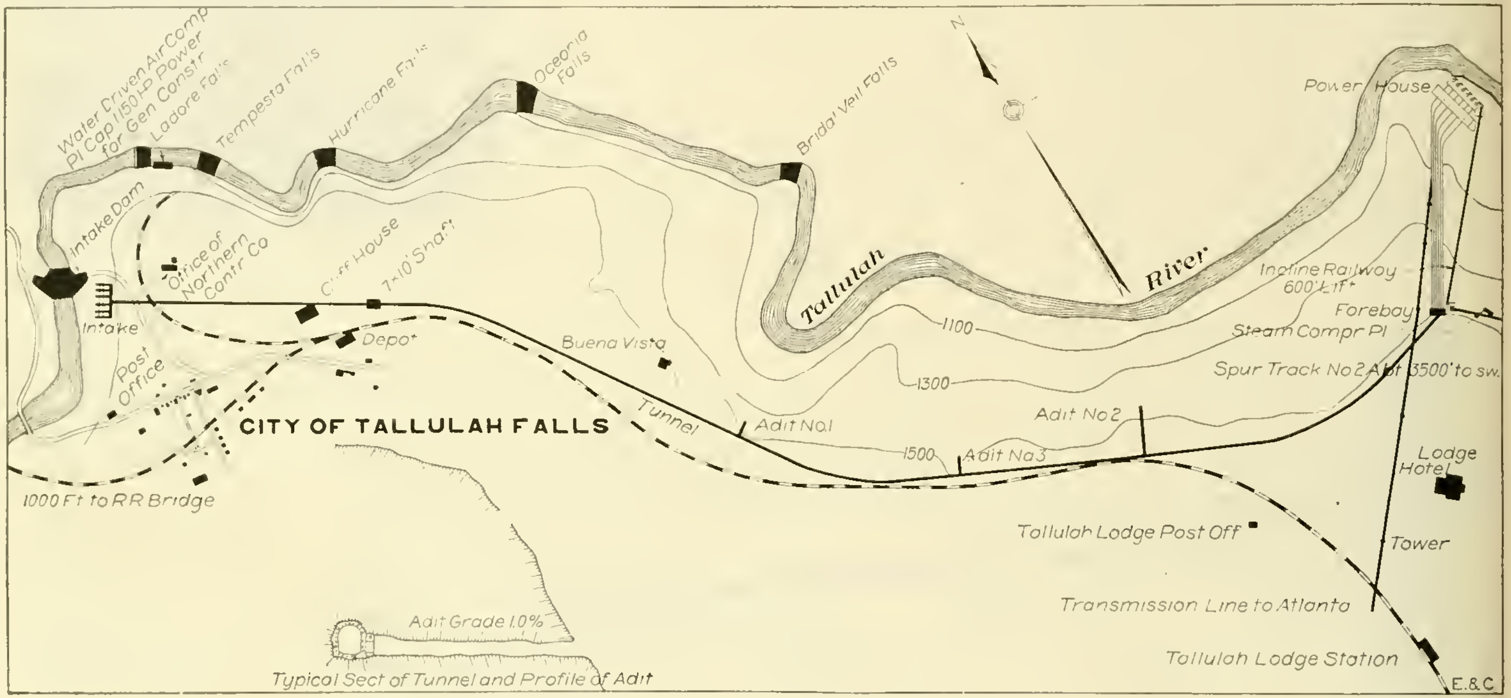


Fig. 1. Plan of Tallulah Falls Development Showing Tunnel Route.

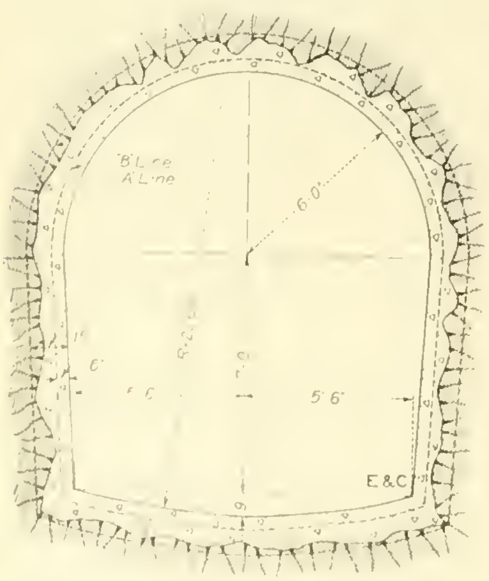


Fig. 2. Section of Tunnel, Tallulah Falls Development.

Figures 1 and 2 give the location, alignment and dimensions of the tunnel, the net area of which is 151 sq. ft. within the concrete lining. A circular section would have been somewhat more favorable to serve the purpose intended, but as no external hydrostatic pressure of magnitude was anticipated, and for the sake of convenience in driving and lining the bore, a section with straight battered side walls was adopted.

The specifications call for a concrete lining having a minimum thickness of 6 ins. (*A* line, Fig. 2) and for the excavation not to extend more than 15 ins. beyond the inside face of the concrete (*B* line). The concrete, a 1:3:5 mixture, was to have a smooth finish, and the material excavated from the tunnel was accepted as being suitable for the concrete aggregate.

The tunnel was driven on a 2 in 1,000 grade, sloping from the intake down to the forebay, through a grey blueish granite, dipping downstream at an angle of 22°, with occasional mud seams. The ground stood well generally, and only 6 per cent of the tunnel length required timbering during construction.

TABLE I.

	Per cu. yd. Stoping work.	Per cu. yd. Bench work.
Labor	$2.415	$2.965
Supt. and walking bosses	.141	.067
General expenses	.036	.101
Office force	.103	.085
Storeroom force	.027	.028
Repairmen	.115	.102
Carpenters	.053	.050
Dynamite	.339	.416
Exploders	.029	.038
Connecting wire	.015	.014
Kerosene	.002	.002
Lubricating oil	.013	.012
Tamping bags	.002	.002
Lamps	.011	.010
Piping and fittings	.015	.029
Drill repairs	.204	.095
Miscellaneous supplies	.041	.063
Drill steel	.107	.080
Blacksmith coal	.027	.027
Freight	.063	.079
Feed	.031	.050
Total	$3.789	$4.315

For the purpose of expediting the work, it was found desirable to drive the tunnel from the intake and forebay ends, as well as from two adits 7 ft. high, 13 ft. wide, 105 and 217 ft. long, respectively, (adits No. 1 and 2) driven on a 1 per cent grade. Before starting work at the intake end, 1,000 cu. yds. of material, mostly rock, had to be excavated. At the forebay, consisting of a shaft 35 ft. wide, 75 ft. long and 70 ft. deep, some 10,800 cu. yds. of rock had to be excavated. Later on, adit No. 3, 150 ft. long, was provided and an 8x10-ft. shaft, 112 ft. deep, was sunk between the intake portal and adit No. 1.

The work under consideration involved the following quantities:

	Cu. yds.
Adit excavation	1,250
Main tunnel:	
Heading	23,330
Bench	29,650
Concrete lining within specified lines	15,430
Concrete lining beyond specified lines	3,540

Seventy-five per cent of the tunnel length was driven with a top heading and bench, and the balance with a bottom heading, the overlaying material being stoped down on the heading floor. Attention is called here to the

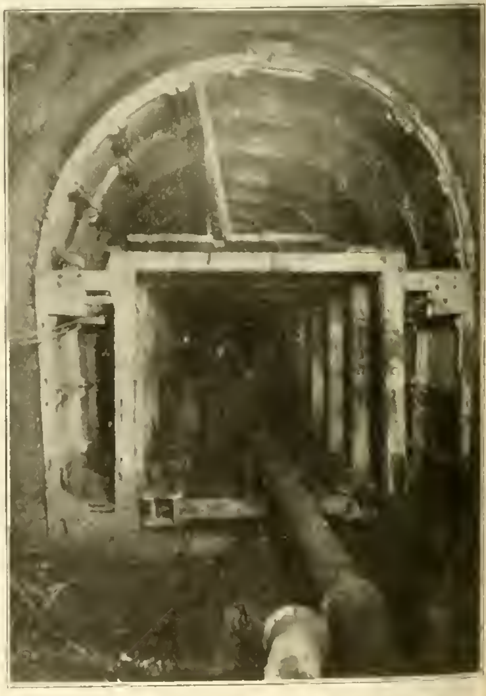


Fig. 3. Steel Forms for Lining Tunnel, Tallulah Falls Development.

fact that, owing to the relatively small section of the tunnel, heading excavation represented about 49 per cent of the total excavation.

POWER PLANT AND EQUIPMENT.

The contractor, the Northern Contracting Co., built a temporary power plant housing wo 500-HP. S. Morgan Smith hydraulic Francis turbines, operating under an average head of 48 ft. and driving two Laidlaw Dunn Gordon air compressors, each having a capacity of 2,500 cu. ft. of free air per minute, 10 lbs. pressure. A wooden dam, 10 ft. high and 60 ft. long, was thrown across the Tallulah River, at Lador Falls, and a 7.5-ft. steel penstock 80 ft. long served to convey water from a masonry intake located at one end of the dam, to the turbines. A 3-ft. diameter steel penstock also diverted water from the intake to a 125-HP. Francis turbine driving a 50-KW. generator used for lighting purposes.

TABLE II.

Items—	Cost per cu. yd.
Labor	$4.18
Overhead charges	0.295
Blacksmiths' repairs	0.107
Machinists' repairs	0.135
Supply house labor	0.043
Office force	0.082
Outside force	0.12
Drill steel	0.06
Blacksmith's coal	0.035
Mules' feed	0.051
Freight	0.144
Drill repairs	.324
Piping and fittings	0.09
Miscellaneous supplies	0.076
Dynamite	0.035
Exploders	0.051
Connecting wires	0.033
Gasoline	0.013
Lubricating oil	0.033
Tamping bags	0.006
Total	$6.913

TABLE III.—TOTAL COST OF TUNNEL EXCAVATION, 39,831 CU. YDS.

February 1, 1912, to April 31, 1913

Items—	Cost per cu. yd
Labor	$3.833
Explosives	.604
Lubricants	.019
Piping	.026
Drill repairs	.172
Miscellaneous supplies	.237
Freight	.087
Transportation	.247
Liability insurance	.181
Miscellaneous charges	.066
Depreciation on equipment	.150
Power*	.306
Total	$[illegible].928

*This item represents that part of the original cost and cost of operation of compressor plant No. 1 chargeable to tunnel excavation.

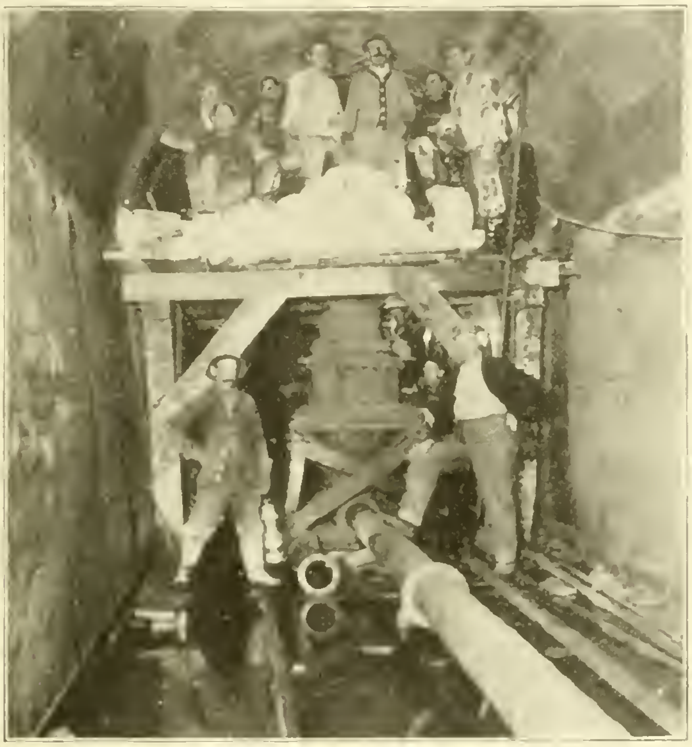


Fig. 4. Charging Platform and Concrete Mixing and Conveying Machines for Tunnel Lining, Tallulah Falls Development.

The tunnel sub-contractor was furnished, free of charge, with 1,000 cu. ft. of air per minute and also with electric current. Owing to low water conditions, in 1912, it was found necessary to increase the capacity of the air plant, and a steam-driven Sullivan straight line air compressor, with a capacity of 1,875 cu. ft. of free air per minute, and two 200-HP. Scotch Marine type boilers were installed near to the forebay and connected with the main air line, consisting of 10, 8 and 6 in. wrought iron pipe, partly laid along the T. F. Ry. track.

Piston drills were used first, but owing to the hardness of the rock and its abrasive properties, necessitating much sharpening, thus being a serious hindrance to progress, Leyner water core air drills No. 7 were adopted, and their use immediately caused a general improvement in progress. With piston drills, starting holes was found a chief difficulty to overcome, and drilling of dry holes gave little satisfaction.

No. 2 Leyner bit sharpeners were installed at each adit and points of attack. At the shaft the following plant was installed:

1 set 10x15-in. Jaw crushers.
1 set sand rolls.
1 60 H.P. steam engine.
1 two-drum 18 H.P. hoist engine operating the shaft cage.
1 set of sand screens.
1 cement house.

TABLE IV.

Items—	Cost per cu. yd.
Labor	$5.12
Cement	2.995
Miscellaneous materials	0.411
Lumber	0.138
Freight	0.066
Transportation	0.157
Insurance	0.168
Royalty on concreting machine	0.420
Miscellaneous expenses	0.106
Crushing rock	2.02
Quarrying rock	0.87
Plastering concrete	0.206
Cleaning up tunnel	0.382
Total*	$12.041

*Blaw concreting forms and depreciation on equipment not included.

Owing to the steepness of the gorge, at the mouth of the adits and intake portal, much of the tunnel muck, necessary for the concrete aggregate was lost, and it was found necessary to open a quarry at adit No. 3.

As a whole, the labor available was extremely poor and unreliable for this class of work. Negro labor was used chiefly. Some Hungarians and Cherokee Indians gave somewhat better results. Rainy weather (annual rainfall varying from 70 to 80 ins.) was a serious hindrance to progress. During holidays a large number of men would leave, thus resulting in onerous transportation charges, and it was no small task to organize and break in two shifts of men for 10 working points. The labor conditions account chiefly for the somewhat slow progress in driving the tunnel.

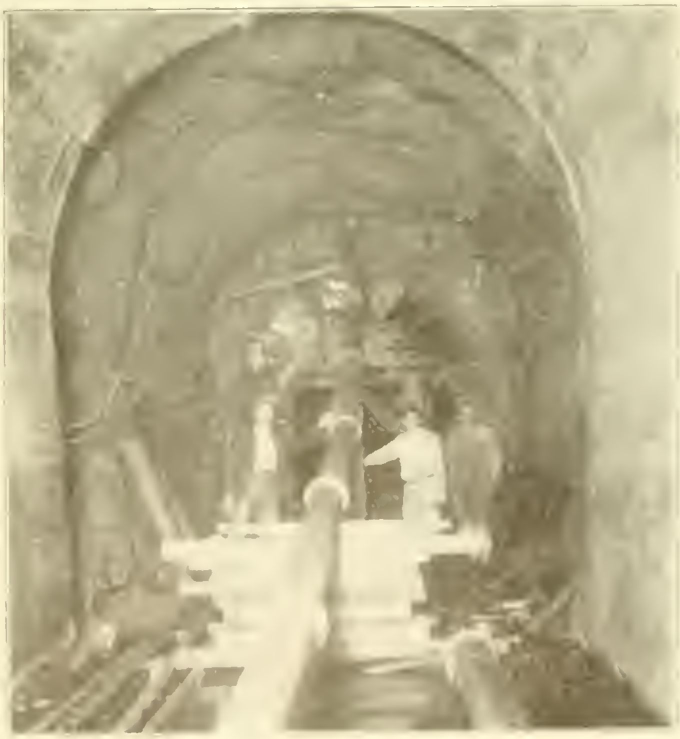


Fig. 5. Delivery End of Concrete Conveying Pipe for Tunnel Lining, Tallulah Falls Development.

Two shifts were worked per day at each

heading, a shift consisting of 4 drillers, 4 helpers, 6 muckers, 2 trammers, 1 foreman. Mules were used for haulage. At the adits, forebay and intake, one blacksmith and one helper did the drill sharpening.

The following wages prevailed: Drillers, $2.50; drill helpers, $1.75; muckers, $1.65; foremen, $4.50; blacksmiths, $3.50; helpers, $2.00; carpenters, $3.00; concrete men, $1.75.

Work at the intake heading and driving of adit No. 1 was started during July, 1911, and work at adit No. 2 on the following month. The intake top heading was first excavated for a length of 800 ft.; at this point ground pressure necessitated heavy timbering and in some instances the roof caved in for a height of 10 ft. Progress was very slow and costly; some water was encountered, and as the tunnel was being driven down grade, pumping had to be resorted to in order to keep the heading dry. It was then deemed advisable to carry the bench excavation close to the heading, and work was suspended pending the completion of the bench excavation.

The headings at adit No. 1 and 2 were carried at the top of the tunnel. A top heading was also started in September, 1911, from the bottom of the forebay, the material excavated being handled with a derrick located at the mouth of same. After the heading had been driven some 500 ft., a soft seam was struck, necessitating timbering.

In June, 1912, after the shaft between the intake and adit No. 1 had been sunk to grade, and adit No. 3 had been driven to the main tunnel, headings 8 ft. high were driven at the bottom of the tunnel section, and the overlaying material was stoped down on the tunnel floor.

The average progress for heading and bench excavation during the year 1912 was 30 and 38 ft., respectively, per week (6 days). Twice the progress was made in stoping work as in bench excavation, at a less cost of about 50 cts. per cubic yard. During April, 1912, about 1,784 cu. yds. of bench material were excavated at a cost of $4.315 per cubic yard, and during May of the same year 3,807 cu. yds. of rock were stoped down at a cost of $3.789 per cubic yard, or at a lesser cost of $0.526 per cubic yard. The cost of bench and stoping work was as per Table I.

A mucking machine which had previously been working satisfactorily in coal mines, and which had been redesigned and somewhat improved for this particular class of work, was installed in the forebay end of the tunnel by the sub-contractor, for the purpose of handling bench material. Besides using much power and attendance, not to speak of maintenance charges, this machine did not prove much of a success and was soon taken out of the bore.

The cost of driving 830 lineal feet of heading (2,856 cu. yds.) was as given in Table II.

The cost of excavating 39,831 cu. yds. of tunnel, from February, 1912, to April, 1913, was as given in Table III.

CONCRETE LINING.

Work on the lining was not started until September, 1912, i. e., at a time when the tunnel had been practically completely excavated. About 120 ft. of tunnel invert was concreted first at adit No. 3, and Blaw collapsible steel forms were then erected and concreted. It was soon found out that it would be preferable to concrete the side walls and arch first, and later on the invert, and this procedure was then followed throughout.

A total length of 240 lin. ft. of Blaw forms were used, the lining being carried on simultaneously at three points. Three concreting machines furnished by the Concrete Mixing & Conveying Co. of Chicago were installed and operated by air at 100 lbs. pressure. The best results were obtained when conveying concrete to the steel forms, erected in 20 ft. lengths only, through 6-in. diameter spiral pipes. The concrete was a 1:3:5 mixture, the aggregate being not over 2 ins. in size. In using these concrete conveying machines, great care had to be exercised in order to prevent the formation of voids within the lining, as its thickness was relatively small. In general, it was found that a somewhat better finish would have been obtained had the lining been given a greater thickness, as it was somewhat difficult to clean thoroughly the forms after these had been used. However, the results obtained were satisfactory for the purpose intended; in wet places the concrete was somewhat honeycombed, but this defect was corrected during the grouting process.

In places where the tunnel roof was high, it was found cheaper to use concrete rather than spalls for back filling, inasmuch as all voids were to be grouted.

The average progress of concreting varied from 30 to 60 ft. per week (6 days), the average for the whole tunnel being about 60 ft. The invert was laid without air concreting machines, as it was found that, in order to obtain satisfactory results, the concrete had to be delivered in a confined space. The invert was laid at a rate of about 745 ft. per week.

The cost of the concrete lining is given in Table IV. Cement was sold by the Northern Contracting Co. to the sub-contractor for the sum of $1.80 per barrel.

GROUTING.

The specifications called for a mixture to be used for this purpose, consisting in 1 to 1½ part sand to 1 part cement. Grout and vent pipes 1½ ins. in diameter were provided in the tunnel arch, or in other places where necessary, 15 ft. apart, more or less. Four grouting machines were used for this purpose, under 40 lbs. pressure. Little trouble was encountered, although in a few places local flaking of the lining occurred where voids had been left. In wet places the grout oozed through the honeycombed concrete, thus making a somewhat rough surface, which was smoothed up later up. The cost of grouting was about $1.10 per cubic yard of concrete placed.

The Tallulah Falls Hydro-Electric Development, including storage dams, power plant, transmission lines and sub-stations is being built by the Northern Contracting Co., Mr. H. M. Atkinson, President and Chairman; Mr. G. V. Brine, Vice-President and General Manager, for the Georgia Railway & Power Co., Mr. P. S. Arkwright, President; Mr. Chas. O. Lenz, Chief Engineer. Mr. Ralph Cadwell is superintendent of construction for the Northern Contracting Co., and Mr. Murray Blanchard was in charge of the instrument work and of the concrete inspection in the tunnel. Condon, Graham & Millner of Knoxville, Tenn., were sub-contractors for the tunnel.

Drainage and Irrigation

Outline of River Regulation and Control in Antiquity.

In his recent address before the National Drainage Congress Sir William Willcocks, the directing engineer of the works for irrigation use of the waters of the Nile and of the works which have restored the Euphrates to its old channel past Babylon, touches with attractive color some points of the history of man's direction of these rivers. Sir William is the builder of the Assouan dam and of the Hindieh dam, both among the most modern of modern irrigation structures, but his account of the dikes and dams of the Nile and the Euphrates built centuries ago by the rulers of Babylonia and Kem sets these works of antiquity in a light which for a moment dims even the technician's view of the greater works of present times. We have space to give only parts of the English engineer's address but they indicate well its qualities which held the interest of those who heard it:

The earliest civilized communities of which we have any knowledge lived in the practically rainless deltas of the Nile and the Euphrates. The early inhabitants of the Euphrates valley called their own cultivated lands "gardens of Eden." Of the Nile valley they spoke as "the garden of the Lord, the land of Egypt." The success of their agricultural undertakings depended then, as they do now, on perfect irrigation and complete control of the rivers and though basin irrigation in one and perennial irrigation in the other were their greatest achievements, the control of the rivers was mastered with no weak hand. Early in the world's history, Noah's flood had taught men that though they could say of their rivers that when the good gods had dug them out, they had set prosperity on their banks, still these same streams were terrible instruments of punishment to those who failed to combat successfully the dragons which lived in their waters.

The earliest river control of which we have any certain knowledge was the work of Marduk, the greatest of the Babylonian deities. The Babylonian epic of creation describes how Marduk first overcame the dragon and then mastered the Euphrates:

> Flowing wide like a sea was the river, when
> Eridu was made, when E-Sagil was built;
> E-Sagil, in the midst of the fresh water deep,
> Where the god of the glorious abode dwells.
> Marduk laid reeds in the face of the waters,
> He piled up earthen banks sheltered by the reeds,
> That he might cause the gods to dwell in the place of their hearts' desire.

Sheltered by solidly constructed and well-protected banks, lay the garden of Eden, a wide stretch of alfalfa, planted thick with date palms, between which were festooned luxuriant vines. The trees of life were there sheltering the garden and the forbidden fruit of the vine. The garden was irrigated by free flow through openings in the dikes. This free flow irrigation is in Genesis mistranslated *mist*. The primitive gardens of Eden, stretched along the banks of the river, and, protected by dikes, prevented the floods from reaching the pastures where the shepherds tended their sheep. The disputes between the agriculturists and the shepherds were as old as the feud between Cain and Abel. The bursting of a dyke destroyed Cain's crops, but benefited Abel's pastures, and on such occasions the Lord had respect to Abel's sacrifice, but had not respect to Cain's.

The ancient Babylonians completely controlled the Euphrates by means of powerful escapes into depressions in the Arabian deserts. These depressions covered 650 square miles and were 35 ft. deep when full of water. In addition to these reservoirs they had the low-lying Pallacopas branch of the Euphrates which took off from above Babylon and discharged the waters of the river into the Chaldean marshes. The first public work Alexander the Great undertook in Babylon was the excavation of a new head on solid ground for the Pallacopas, known some years ago as the Hindieh branch, and today the main stream of the Euphrates. Its head had hitherto been in sandy soil, and as the branch had to be opened in very high floods to escape the excess waters of the Euphrates and then immediately closed after the flood to keep the main stream full of water past Babylon, the closure had been a work of extraordinary difficulty entailing the presence of 10,000 men. Next to building a masonry barrage this was the wisest thing he could have done.

The lands of the lower Euphrates which were exceptionally fertile and supported a dense population were protected by banks 100 ft. wide. Their remains can be followed today along hundreds of miles.

Provided with escapes into depressions in the deserts whose contours can be followed for scores of miles by the presence of thick belts of Euphrates shells, with regulation at the head of the Pallacopas branch, and with broad and majestic dikes well protected by brushwood, the Euphrates was thoroughly controlled, and we do not read of any disastrous inundations on its banks. The lessons learned during the terrible flood of Noah's time had been engraved on the minds of the Babylonians by a diamond pen. This local flood on the Euphrates has been magnified into a world-wide disaster by the mistranslation of a single word. In the ancient, as well as in the modern, language of Mesopotamia, "desert" and "mountain" are expressed by the same word.

The Tigris enters its delta at Beled, north of Bagdad, over the ruins of one of the most interesting works of antiquity. In ancient days, some giant, local tradition says Nimrod, barred the valley by a massive earthen dam, raised the river some 40 ft. and turned it over the hard conglomerate, so that it could flow at a high level and irrigate the whole country. The left-hand canal was the Nahrwan which had three heads, was 400 ft. wide and 18 ft. deep, and had a length of 250 miles. This work existed for over 3,000 years and was only swept away in the time of the last feeble Caliphs.

Though the ancients harnessed the Tigris for irrigation, they never controlled the floods of this river as they had those of the Euphrates, and there were occasional inundations when the dikes were burst, which spread such ruin and disaster that no records exist of certain years except the dismal tale that the Tigris had burst its right bank. The whole right bank of the river was protected by a broad canal with high banks, known for the greatest part of its course as the Nahr Melcha or the Royal Canal. The most powerful kings of Babylonia were proud to record the fact that they had strengthened and fortified these banks. The Tigris floods relieved themselves by overflowing the left bank of the river which was abandoned to them.

The perfect control of the Euphrates where massive dikes were supplemented by escapes into the deserts, and the imperfect control of the Tigris where double banks without escapes into the deserts failed to provide absolute protection, teach the lesson that protecting dikes unaided by works of control higher up the stream are not of themselves sufficient to insure the lower reaches of rivers against inundations in years of exceptional flood.

Cyrus the Great controlled the Gyndes, a tributary of the Tigris in a truly original manner. Babylonia was already peopled and lands were needed for his Persian troops. The Gyndes discharges 40,000 cu. ft. per second and runs 30 ft. deep in a sandy and mobile bed. He could build no regulator, and so he dug thirty canals, divided the waters of the river among them, closed the river by an earthen dam, and completely controlled it. It was a striking example of the truth of the proverb "Divide et impera." As he could never have induced his wild soldiers to dig these canals for any useful purpose, he took advantage of the fact that his favorite horse had been drowned in the flood and urged his soldiers to dig the canals and dissipate the waters of the river in such a fashion that it could never again drown a horse. One of the canals bears even today the name of "Khorassan" from the far distant province in the east of Persia, which supplied the contingent of troops which had dug it.

Alexander's historians record that before his navy could sail up the rivers, he had to have the dams across them removed. Such dams on considerable streams were in those days made of earth, and they consequently completely cut off the supplies. Every drop of water at the time of low supply was used up for irrigation. But not only were the waters of the main streams like the Tigris and the Euphrates, and of secondary rivers like the Gyndes controlled by earthen dams, but small tributaries like the Adhem were barred by imposing masonry dams where they issued from the hills, and the water led off into canals excavated on both banks. Whatever water issued from the Persian hills into the delta of the Tigris and the Euphrates was utilized for irrigation. Alexander was so struck by the possibilities of the delta that he was planning the rebuilding of the tower of Babel and the making of Babylon the capital of his world when death overtook him, while he was personally conducting the drainage of the Chaldean marshes.

In the neighboring kingdom of Persia, masonry dams at Shuster and Ahwaz on outcrops of sandstone on the Karun River and opposite Ahwaz on the outcrop of the same rock on the Choaspes, the River of Susa, controlled those streams and allowed of the summer supply being utilized for irrigation. Each of these streams discharges over 120,000 cu. ft. per second in flood.

The Nile in flood does not carry one-fifth the sediment that the Tigris or the Euphrates does, and consequently the transverse slopes in Egypt away from the river are only one-fifth as steep as those in the other delta. If the Nile breaches it banks in flood, it can be brought back after the flood to its old channel without any serious difficulty, while a very severe breach on the Tigris or the Euphrates has been followed by the river completely leaving its channel and forming a new one many miles away. Though flood protection in Egypt is not of such imperious necessity as in Mesapotamia, it is still a matter of no small importance, and its great value did not escape the able men of the ancient world.

Menes protected the left bank of the Nile with dikes and probably treated the right bank of the river as the Babylonians had treated the left bank of the Tigris. It was Amenenhat of the 12th dynasty who put the whole Nile valley under cultivation, constructed the dikes on both banks, and provided the Nile with an escape into the wide and deep depression of Lake Moeris. This escape was one of the seven wonders of the Greek world. They led the flood into the depression when it was dangerously high and provided for its return to the river when the inundation had come to an end, so that the reservoir might be empty when the next high flood came. The gigantic dikes of entry and exit were only cut in times of emergency and were reconstructed again at an expense of labor which even builders of pyramids considered excessive. By cutting the dike in a year of ordinary or low flood, the level of the Nile in lower Egypt could be lowered to such an extent that a famine became inevitable, and it was the capture of the dike by the king of Upper Egypt which produced Joseph's famines and its recapture by the king of Lower Egypt which brought them to an end. The Nile shells which mark the limits of the ancient Lake Moeris tell their tale like the shells which lie round the Babylonian depression.

On the approach of the red water of the Nile flood the weaker branches of the Nile were closed by earthen banks some distance above their mouths to allow the flood to freely inundate the country, and it was the shutting off of the fresh water and the entry of the sea water into the tail reaches which killed the fresh water fish in myriads just as the red flood came, which misled unscientific observers into thinking that it was the red water which killed them. And indeed it was local knowledge and skillful manipulation of these same means which enabled Moses to extricate the Israelites with a high hand and overwhelm Pharaoh's host in the Serbonian bog or the "River of Reeds," mistranslated the "Red Sea."

The lessons taught by the Nile center round the fact that all reservoirs constructed for flood control must be empty at the beginning of a flood, so that they may be fully available for withdrawing water from a dangerously high flood. Moreover, if they are not to be obliterated by silt they should be only used when absolutely necessary.

Between the second and first cataracts the Nile is trained by gigantic spurs which advance boldly into the stream and insure a deep channel for navigation in the lowest stages of the river. These spurs have been constructed with care and have top courses of roughly dressed stone which have stood for thousands of years.

Egypt is practically free of malaria and the Euphrates delta is not scourged by it, but there is one valley in the old world which has never been fully developed owing to its prevalence, and that is the valley of the Jordan. The waters of the Jordan are too low for irrigation, but the valley is full of beautiful and copious springs, which are generally more of a curse to the people than a blessing, as wherever there is a spring of water there are mosquitoes and malaria of a very malignant type. That the ancients had some idea of the value of fish in keeping down mosquitoes and malaria seems likely from the fact that the fish god Dagon was worshipped in the low-lying malaria haunted valleys and plains. One can easily see how it was that before Joshua's crossing of the Jordan, a severe earthquake displaced a spongy shoulder of Mount Gilead, which slid across the valley, while Jordan fled and the mountains skipped like rams and the little hills like young sheep, and the valley of Jerico crumbled on their base.

The lessons to be learned from the river regulation and control of the ancients are writ large over all their undertakings. Thoroughness, combination and continuity. The further we go back in the world's history the more thorough was the work. They designed and constructed like men who realized that they had thousands of years before them. While divergent interests had kept them apart they had so often drowned each other out in one valley and starved each other out in the other that self-preservation made them combine. The world moreover was young then and the works were pursued year after year and decade after decade without any racing against time. And it is extraordinary how effective and durable were their undertakings. Their works so struck the imagination of after generations that these latter could only conclude that there had been giants in the earth in those days.

Organization of Irrigation Operating and Maintenance Force and Its Training.

The methods of managing an irrigation system based on practice on the great Canadian Pacific R. R. was discussed in a notable article in our issue of Jan. 28, 1914. A somewhat similar discussion by George H. Bliss, Project Manager, U. S. Reclamation Service, Boise, Idaho, before the Idaho Society of Engineers is published here:

In the history of all irrigation projects that are carried on to completion through the construction and into the operating period, there are three distinct stages of development, each with its peculiar problems.

The first period covers that for which the problems are mostly of an engineering nature, and includes the period of preliminary and final surveys and construction work. During this period no water is available for distribution, and such farmers as may have already located upon their homesteads find ready employment upon the construction features of the project, or are occupied with establishing themselves on their farms and in getting ready for water during the seasons to follow. To carry on the work successfully during this period, the force of men who will prosecute the field work will be those with an engineering education and experience.

There will follow this period one during which some of the upper units of the project will receive water while the lower units of the project are still in the construction stage. Many human problems will begin to appear during this period, and different types of men must be developed or obtained to handle these problems successfully while the engineering work in the lower sections of the project is still being prosecuted.

During the third period the construction features have largely disappeared, except for betterments and extension work, water is available for all sections of the project, the construction work on which the entrymen have been employed is completed, and the farmer turns his attention to his farm and tries to develop it with the resources at his command.

The problems that the management must contend with from this period on are mostly those into which the human element largely creeps, and the amount of engineering work and the need of men with engineering training will gradually diminish. It is with this third period in connection with an irrigation project that this paper will deal.

- MANAGER
  - ENGINEER AND ASSISTANT MANAGER (Not needed on small projects)
    - CLERICAL DEPT.
      - This department will handle all clerical matters, including correspondence, files, collections, disbursements, purchases, bookkeeping, and accounting. On smaller projects the chief clerk may also be the fiscal and purchasing agent. On larger projects the fiscal agent, purchasing agent, bookkeepers, storekeeper and stenographer will report to the chief clerk.
        - Water record clerks: Keep all records of water deliveries and status of water users, telephone messages to water master's offices, and receive messages pertaining to water deliveries.
    - ENGINEERING DEPT.
      - Assistant Engineer
        - Draftsmen: Design all structures not standard, and adjust standard designs to meet local conditions. Are responsible for all mapping, file designs, maps and profiles.
        - General Foreman: Needed at times on large projects to handle construction work of such a scope and character as not to be readily handled by the subforeman reporting to the water master.
        - Hydrographer: General supervision over everything pertaining to the measurement of water, measuring devices, automatic records and alarms; rates all measuring devices, and prepares tables.
          - Inspector: Visits all patrolmen and inspects their work, reports needed changes in measuring devices, and foul condition of weir pools and ditches. On small projects may be the hydrographer.
    - OPERATING DEPT.
      - Water Masters
        - Water Masters: Have direct charge and are directly responsible for everything pertaining to the delivery of water and care of system in their divisions.
        - Ditch Rider: Delivers water to the users, keeps the records of water deliveries, patrols and keeps in repair about 18 miles of canal.
        - Subforeman: Handles general repair and betterment work, such as can be undertaken by the water master in connection with his other duties.
        - Clerk or Timekeeper: In office of the water master day and night, answers phone calls, receives and transmits messages and complaints, keeps time and record sheets.
      - Telephone Lineman and Troubleman.

Although during the third or operation period the engineering problems have greatly diminished in number and extent, they have not wholly disappeared. Many of the structures which have been installed will need to be readjusted best to serve the object for which they were installed, additional and more adequate wasteways will be required, additional structures must be designed and installed, and drainage ditches must be located and constructed, all of which work calls for engineering ability.

On the smaller projects the manager may personally be able to attend to a great deal of engineering detail, but on the larger projects this work must largely be entrusted to an assistant, who in many instances will also act in the capacity of an assistant manager. Such men can be selected from the engineering forces that have prosecuted the construction work during the earlier periods, and the fact that they have been long employed on the project and have followed through the various stages of its development should qualify them to continue the extension and betterment work and later take up the management of the project.

The real problems of the operating period are those of adapting the system and the organization to the needs of the people who are on the farms and who may be either made or ruined in the degree that the system is properly or improperly managed, and of adapting the men on the farms, who have come from all localities and walks of life, many without any experience in irrigation farming, and some without any farming experience at all, to the irrigation system that the manager is called upon to operate.

In the early operating period there will be much to try the patience of both the farmer and the manager; ditches will be new and breaks may be frequent; additional structures will be found necessary, and some that have been installed will be found to be inadequate or improperly located; all of which will cause annoyance.

MANAGER.

The manager should arrange to spend as much time as possible in the field looking after the interests of the water users and the organization that he is managing. Much friction will be overcome by quick action in the field, and all cases of serious complaint should be looked over carefully on the ground with all interested parties present before reaching any decision. When in his office the manager should be accessible for consulting with the water users at all times. A feeling of cordiality between the manager and water users will go a long way towards settling disputes. The manager should meet with the water users at their regular meeting places to listen to complaints and to explain the policy of the organization. He should insist that his associates in the organization be energetic and industrious, non-partisan and fair-minded at all times.

ASSISTANT MANAGER.

The assistant manager who will be required on large projects acts for the manager in his absence, and at other times will handle the details connected with engineering and construction work. He should listen to any and all complaints that the water users have to make and bring them to the attention of the manager.

WATERMASTERS.

On the smaller projects the manager or assistant manager may be able to give the details of operation and maintenance close personal supervision, and the ditch riders and others engaged in operating and maintaining the system may report directly to them. On the larger projects, however, it will usually prove advisable to divide the project into two or more divisions, placing a watermaster or superintendent in charge of one division of from 30,000 to 40,000 acres of land.

A good watermaster is a valuable asset to any project, and the position is one that it is hard to fill satisfactorily, as comparatively few men, either on account of their training or temperament, are qualified to fill it. The watermaster must be a man of good judgment, with experience in handling successfully work of a large nature, such as is gained in construction or engineering work. He should also have experience in farming in order that he may be able to recognize the difficulties that are being experienced by the settler and assume a sympathetic attitude. The attitude or disposition of the watermaster and his ability to cope with conditions and difficulties and to bring about a satisfactory and harmonious solution will go a long way towards making any project a success. The watermaster is the man on the ground with whom the water users come more directly in contact, and his relations to the water users should at all times be friendly and cordial. Numerous complaints, many without much foundation, will come to the watermaster, and he must give them respectful, careful, and judicious consideration. Combativeness in a watermaster should not be tolerated, as the water users who give him the most trouble are the ones often times with whom he must spend the most time. The tension under which a responsible watermaster for a large system will work during the operating season is severe, as night or day he must be prepared to move quickly to meet the emergencies that arise.

Farmers who have the desire and ability to handle construction work, those who have been construction foremen during the earlier periods of the project and have a working engineering knowledge, will if they have the proper temperament make good watermasters, as will also men raised on farms who have taken an engineering or agricultural course in the State universities. Often a ditch rider who has shown ability will be qualified to become an assistant watermaster and from that position will be promoted to that of watermaster. Strength physically and mentally is required to fill the position satisfactorily, together with the faculty of dealing pleasantly with men even when under a severe strain.

DITCH RIDERS.

The watermaster will have reporting to him a corps of ditch riders or patrolmen, their number varying with the size of the system and the method of operation used. If the laterals are patrolled and deliveries are made to the farm units, one ditch rider will patrol on a circuitous beat from 16 to 20 miles a day and make from 30 to 40 deliveries. Except at times of heavy wind storms the rider will be able to keep his canals free from weeds and in a good operating condition and should be required to do so.

The ditch rider if he visits every turnout each day will come in closer contact with the water user than the watermaster, and a good deal of thought should be given to his selection. He should have a knowledge at least of the principles of agriculture in order that he may have the proper attitude towards the farmer. He must also have a good common school education and be neat and accurate with his records. A great deal of trouble and friction can be caused through a ditch rider who is improperly constituted. Good judgment and impartial treatment in connection with the water users must be demanded. An indifferent, careless, or irresponsible attitude should not be countenanced.

Good canal riders can be found in almost any locality among the farmers or the older farmers' boys. Ditch riders under 25 and over 50 years of age are not usually desirable. A rider should be strong, energetic, resourceful, and tactful and must be able and willing to perform hard manual labor in case of breaks or when employed on maintenance work. It is advisable that the ditch rider be experienced in miscellaneous construction work.

The best results will be obtained where the canal riders are kept permanently on the same beats, and this is especially so if during the non-irrigation season the rider is allowed to manage the maintenance and betterment work covered by his beat. The rider should therefore be selectd with the idea of permanency in mind, and so far as possible riders should be selected who have qualifications that will permit of advancement to higher positions.

The fact that a rider owns land under the project should not disqualify him for the position. Other things being equal the ownership of land and his acquaintance with the problems of the farmers should tend to increase rather than decrease his usefulness as a canal rider. It is best, however, for obvious reasons not to assign a rider to a beat from which he will deliver water to his own land, or to his relatives or neighbors.

The rider should be required to give his entire time to the matter of canal riding, and not be allowed to devote a portion of his time to farming and private work. Loitering should not be tolerated, and a canal rider should be made to feel that he is employed for the sole purpose of delivering water impartially to the water users along his beat, of making accurate records of those deliveries, and of keeping the canal patrolled by him in good operating condition.

A rider should note and report complaints made by the water users, and in receiving such complaints his manner at all times should be courteous.

In order that riders may do efficient work, written instructions should be furnished them and such tables as they may need. They should be trained to be watchful of the canals and structures on their beats, of the telephone system, and all other features of the project within their territories, and to be careful in all things connected with the successful operation of the system. Whenever possible frequent meetings between the manager, water masters and ditch riders should be arranged for the purpose of discussing problems encountered in operation and maintenance work, as well as policies and methods to be adopted.

AGRICULTURAL EXPERTS.

On most projects it may be advisable to have employed one or more agricultural experts to assist the farmer on the farm and in his farm life. Such men if employed should be carefully selected and must be men well qualified by training, experience, and temperament to make a success of work of this nature. Men amply suited to this class of work can usually command good salaries, and many projects cannot afford to employ men of this character. A less expensive man or one of less experience would probably prove of little value and might actually be injurious to the best interests of the project. In case an agricultural expert of the proper sort cannot be employed, much good can often be accomplished by the manager and his associates by calling the attention of interested water users to the results obtained along certain lines by successful farmers on a more remote portion of the project.

CLERICAL DEPARTMENT.

Besides the engineering and operating departments, all projects must maintain a clerical department commensurate with the size of the project. On the large projects this department will be in charge of a chief clerk. On the smaller projects one or two general clerks may suffice. Routine matters and correspondence, water records and statistics, collections, bookkeeping and accounting, all call for clerical assistance, and much confusion will be avoided on a large project by having a well organized clerical department. The correspondence will be opened and distributed to the various departments by the chief clerk, and much of the routine correspondence can be answered by him directly, which as a rule will result in a saving of time and more efficient service.

These three departments, engineering, operating, and clerical, will complete the organization to carry on the operation of a large project, and if care is given in the selection of the men in charge of the departments, should prove to be an effective organization.

New Cement Works for India.—The Kashmier Iron Mines & Power Syndicate has obtained a concession from the Punjab government in India, and will enter into the business of manufacturing and selling Portland and other cements, limes and plasters.

Typical Inclined Drop for Irrigation Canal, Milk River Project, U. S. Reclamation Service.

(Staff Article.)

A series of five drops in the St. Mary's Canal of the Milk River irrigation project differ structurally somewhat from earlier designs of the Reclamation Service engineers. These drops are all of the same type, and drawings of one alone are sufficient to illustrate their construction. Figure 1 shows a half plan and the essential sections; all construction is reinforced concrete. The general structural features that are to be noticed are: (1) The cross-channel wier at the inlet; (2) the converging of the sides of the chute as it descends the hill, and (3) the vertical drop of the chute into the surge basin. The hydraulic properties of the channel of the drop are as follows

Section.	A	V	Q.
Inlet canal	396	2.23	883
Sta. 1422+03	37½	22.67	850
Sta. 1422+49	28½	29.6	850
Sta. 1423+35	24	35.4	850
Outlet canal .	550	1.55	850

It will be noted from these figures that no effort is made to retard or break up the stream during its descent of the chute; rather the velocity is increased by a converging channel and a surge basin of good size is depended upon to still the current. The relative dimensions of total drop and depth and length of surge basin are given by the drawings, as are all the structural details. Concrete materials and construction conform to the specified standards of the Reclamation Service. The engineers in immediate charge of the Milk River Project are H. N. Savage, supervising engineer, and Joseph Wright, construction engineer.

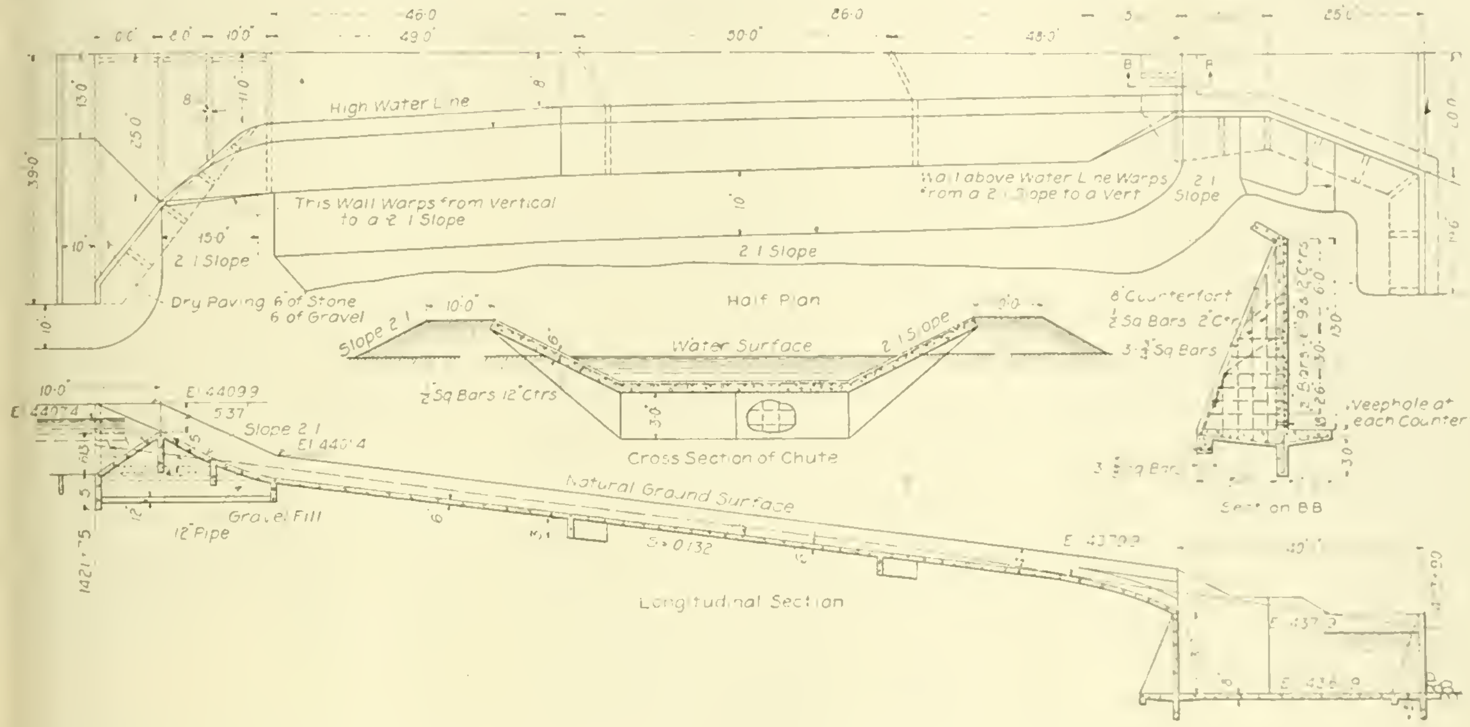


Fig. 1. Inclined Drop for St. Mary's Canal, Milk River Project, U. S. Reclamation Service.

Announcement is made of the fourteenth annual six weeks summer school of the College of Engineering of the University of Wisconsin which opens on the 22 of June. Courses of instruction and laboratory practice are offered in electrical, hydraulic, steam and gas engineering, mechanical drawing, applied mechanics, testing of materials, machine design, shopwork and surveying, in addition to which courses may be taken in the College of Letters and Science. For bulletin address F. E. Turneaure, University of Wisconsin, Madison, Wis.

Roads and Streets

Proposed System of Cost Accounting for the Highway Department of Los Angeles County, Calif.

The recent passage of laws concerning road work in Los Angeles County, Cal., will probably permit a centralization of the administrative and executive work of the highway system of the county. A recent report of Mr. John C. Veenhuyzen,, on a cost accounting system for the highway department is given here in part.

The funds of the county are divided under three heads: (1) general fund, (2) road fund and (3) highway maintenance, and the cost accounting system is worked out with these in mind. In the matter of keeping a check on these funds the report states that "it must be taken into consideration that while the road department will have an accounting system on the accrued basis founded on payrolls and invoices, the auditor's accounting system is based on records at his disposal, which are payrolls approved and demands allowed."

In connection with interest charges and the capitalization of plant and equipment the report continues:

As the improvement of all roads is solely for the purpose of improving the service of such roads, it is necessary to include the interest on the investment as an expense burden in order to show if the return in better service is adequate to the total expense incurred for such service. There is no way to compare the value of several improvement systems as to their economical efficiency, unless each road unit is properly capitalized and burdened with the proportionate charge of interest on the investment, as well as with expenses resulting from maintenance and repairs.

The interest, although paid out of a different fund and as such being a cash expense, has to be earned by the road in some manner, either by better service or less expensive maintenance, and although this earning will not show as an actual revenue but merely as a transfer account, it stands for proper accounting to show all elements of the actual cost of maintenance in a comprehensive manner and in collective figures that can be grasped and understood by every one without the necessity of compilation from scattered sources of information, which are all true and correct in detail, but whose relation to each other can only be grasped by the expert. This principle holds equally true for all investment of county money either in buildings or equipment, and for that reason should be applied in the cost accounting of the Pacoima quarry, county oil pit and all other permanent improvements. To illustrate the above, I call attention to the fact that in the accounts of the highway commission the Pacoima quarry appears having a value as of Jan. 1, 1914, of:

Construction	$38,064.05
Equipment	55,130.70
Total	$93,194.75

It is at once apparent that one element of valuation is missing, viz: Lands or ground upon which this institution is standing. This land was acquired by the county and paid for with $22,891.25 in June, 1909, but being paid out of a different fund it appears under a different classification, with the result that correctly the value of this quarry as of Jan. 1, 1914, should be $116,086, but it requires compilation from scattered sources to arrive at that value. In attempting a cost accounting for the Pacoima quarry, it is this value of $116,086 which will have to be considered as the investment on which the service of the quarry has to show results. The county is entitled to earnings on that investment, and not receiving any real revenue from it, we must show that there has been enough saving in more efficient service or in cheaper production of material over that bought in open market to make up for the loss of revenue which this investment value could produce in interest.

ORGANIZATION.

The prime factor of organization is the logical grouping of work under functional activities and the establishment of definite responsibility and authority. This plan is best formulated in the shape of graphical charts. The entire work of the road department may be summarized as follows: (1) general administration of the affairs of the department, (2) design and construction of highway improvement, and (3) maintenance or care of improvements already made. A diagram of their organization is shown in Fig. 1.

General Administration.—The division of general administration embraces the functions of accounting, purchasing (or requisitioning same), store-keeping, appointments, correspondence, stenographic service, secretarial service, etc.

Engineering.—The division of engineering embraces topographical work, design, construction, sub-surface structures, improvement proceedings, clerical service and testing laboratory if a separate laboratory for this purpose is needed, and a transportation division to arrange for the supply of vehicular service, both passenger and freight.

Maintenance.—Under the maintenance division are consolidated all of the maintenance forces of the department. The organization contemplates a territorial division into 36 sections, each in charge of a road foreman. The Pacoima quarry and the county oil pit will be functions of this division.

In putting this plan into actual operation, the policy should be to do so with as little interruption to current work as possible. The reorganization of inter-related work, such as, for instance, bridge work, demands that advance must be made step by step until full control can be established.

In order to proceed with the reorganization, certain physical changes will be found necessary, re-location or establishing storerooms and warehouses, re-arrangement of road districts as territorial sub-divisions, establishing of road units, etc. With a view to securing knowledge essential to the control of efficiency in maintenance forces, it is necessary to establish a suitable system of cost records and procedure. Field and office conditions will have to be anticipated as far as possible and a definite procedure, governing use of forms and work, should be made in book form for the education of all employes.

OBJECTS OF COST SYSTEM.

The objects of the proposed cost system are as follows:

(1) To ascertain the total and unit cost of each class of work performed in the proper exercise of all functions of the department.

(2) To show the quantity and cost of each class of work performed within the boundaries of any road division and for any separate road unit.

(3) To show separately the three main elements of expense entering into the cost of each class of work as follows: (a) Labor charges, (b) material charges, (c) plant and equipment charges.

(4) To show in a comparative way the cost of performance for each unit.

(5) To secure by interpretation of these data knowledge as to the efficiency of performance by departmental forces, thus facilitating administrative control.

(6) To show when taken in conjunction with records of current conditions the adequacy of service rendered the public.

(7) To permit a determination of the point beyond which it will be undesirable, from an economic point of view, to continue maintenance work on existing roads, or, in other words, the time when replacement must be contemplated.

(8) To provide data on which departmental estimates may be based.

CLASSIFICATION FOR UNITS.

The first problem consists of establishing, on a uniform basis, a complete and logical classification of all functional work in such form as to permit the application of units of measure to each class of work.

In order to meet these conditions the description of work performance can be subdivided under four separate heads as follows: (1) Bureau or general function, (2) Class or specific function, (3) Operation to be performed, (4) Elementary activity involved in the performance of operations.

By means of this arrangement it will be found possible to prepare a list of descriptive phrases under each one of the four heads above mentioned, and in consequence by selecting a suitable combination of phrases from the four lists any performance whatever can be entirely described and completely defined.

This work is perhaps the most important and difficult in connection with the design of a cost system, since a large degree of elasticity in phraseology and terminology is required to include every feature of work performance. The result of this classification will be in the form of a code book wherein the descriptive phrases are arranged in a way that makes it possible to show which of the items under each head are applicable to items under the preceding head, together with a complete code number for the full description. In this way all of the elementary activities that are applicable to the operations to be performed on a given class of work may be readily indicated.

MEANS OF SEGREGATING ITEMS.

The second problem in designing a cost system is selecting satisfactory means by which both direct and indirect charges to classified work may be properly segregated and compiled. This problem may be solved by the adoption of tabulating data recording cards. Inasmuch as the complete cost of any work may be obtained by a summation of the three basic elements of expense referred to above, namely: (1) Labor charges, (2) material charges and (3) plant and equipment charges, three forms of cards should be used, each designed to record one of these elements, either as a direct or an indirect charge, to some class of work or function. In connection with these three forms of cards, another form of card should be used on which is to be recorded the class and quality of finished work performed at a given point. The use of this last mentioned form makes possible the computation of unit class costs and the allocation of maintenance charges to specific territorial locations or road units. A distinction is to be made between "finished work" as fully described by the first three work classification codes and the completed elements of unfinished work, which requires the full four codes. It is obvious that where a job cannot be completely finished within the period covered by a field report it will be inaccurate to use as a basis of comparison the unit cost of that part of the work which is finished, since the actual cost of performance within this period will nearly always include certain charges for some work on some unfinished portion of the job. For this reason it should be planned to have the amount of "finished work" not reported until the full completion of the job, at which time it will be shown by filling out one of the above mentioned "finished work" cards.

The quantity of work described by the elementary activity is always susceptible of measurement within the period covered by the work report and will be shown on one of the cards used to record labor or equipment charges.

The arrangement of such cards provides for the segregation and compilation of any kind of information that will normally be required.

The plan of making charges to work under the cost system rests upon the following basis:

All primary charges will be classified under one of three heads: (1) Labor, (2) material, (3) equipment.

The use of labor, material and equipment must be recorded as directly chargeable either to (1) some measurable work which constitutes a function of the department, or (2) to some auxiliary service, the cost of which can be charged directly to such functional work, or (3) when this is not done to some deferred or suspense account, the total of which will be periodicaly distributed over the functional work. Items treated as described is (2) and (3) will be in the nature of indirect charges or burden on labor, material or plant and equipment.

Labor.—Labor will include all classes of personal service. Charges representing a burden on labor will be pro-rated either monthly or quarterly, as may be established, over the functional work performed on the basis of one of the following principles, as may be determined as best suited: (a) Cost of productive labor, (b) mileage per road unit and a corresponding fixed figure for such bridges as are considered to be separate units, (c) surface area of road units and such bridges as are considered to be separate units.

These figures once established, it is a simple matter to pro-rate overhead expense.

All labor charges, whether direct or indirect, will be made at the payroll rate for the time employed.

Material.—Material will include all classes of expendable commodities which enter into construction, repair, replacement or general care as an element of the finished work, and which, when once used, are not further accountable by inventory.

For cost purposes material should be classified under three general heads: (a) Burdened stock (b) production stock, (c) miscellaneous stores.

Burdened Stock.—Burdened stock consists of all articles used in sufficient quantity or of sufficient importance to warrant separate imposition of burden charges.

Charges representing a burden on such commodities or tending to increase their actual cost at the point of use, must be added to the invoice price.

Charges for the use of such commodities as applied to functional or auxiliary work should be charged to the work at an arbitrary unit price (subject to periodical adjustment), the price being established at a figure which will, when the stock has been consumed, approximately extinguished the purchase cost plus the added burden charges.

Differences between actual cost at the point of use and arbitrary charges to the work should be pro-rated over the cost of material used during the preceding month or quarter.

Production Stock.—Production stock consists of all articles secured or manufactured by the use of the departmental services. Stock thus produced must be treated as though purchased as soon as the actual cost of its production can be ascertained.

Miscellaneous Stores.—Miscellaneous stores consist of articles seldom used or used in small quantities, or of small value and not included in burdened or production stock. Direct charges for the use of such should be made at the unit purchase price.

Burden charges, such as for general care and handling of miscellaneous stores, as well as burden charges on other material that can not readily be allocated to burdened stock must be pro-rated over the functional work on the basis of total prime cost.

Equipment.—Equipment includes all loose and movable property intended for continuous use and accountable by inventory. It also includes all fixed and stationary property in the nature of power-producing or manufacturing plant and machinery. No articles in the nature of supplies which can be classified as material should be included in equipment.

CLASSIFICATION OF EQUIPMENT.

For cost accounting purposes equipment can be segregated into three classes, viz:

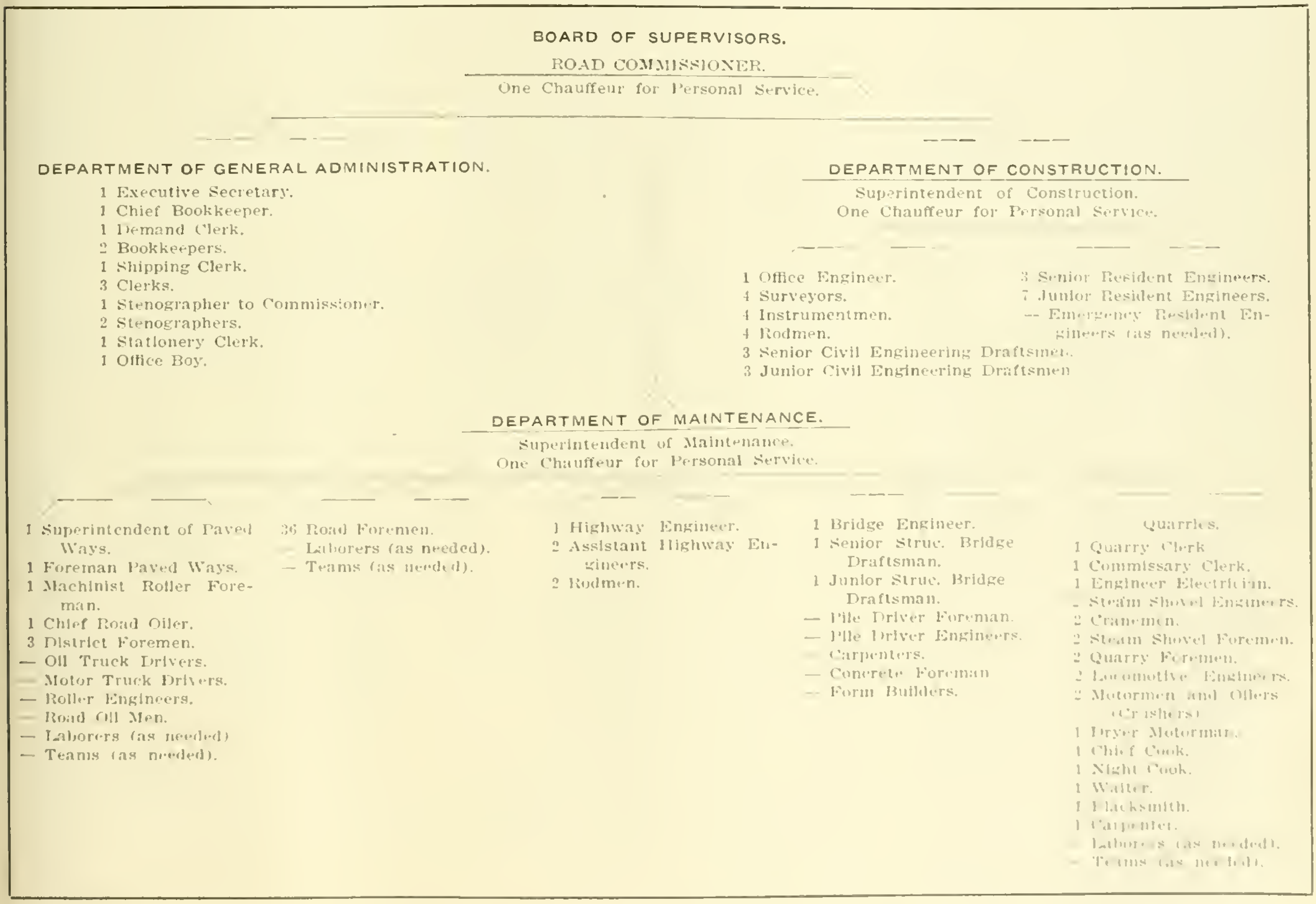
Class "A."—To include all equipment with attached accessories as originally purchased or constructed, the productive operation of which requires the application of self-generated power, or power obtained by transmission from a general source.

Class "B."—To include all equipment with attached accessories as originally purchased or constructed not embraced under Class "A," which has a life of more than one year (excluding parts).

Class "C."—To include all equipment not embraced under the two previous classes, the probable life of which is, or may be assumed to be, less than one year. It also includes loose tools and implements, small parts and accessories controlled by the road foreman, as described later.

Charges for the use of Class "A" equipment should be made at an arbitrary rate per hour to cover the cost of maintenance and depreciation only. No operating costs will be included in charges for this class of equipment, inasmuch as the operation will always be coincident with its productive use and the cost of operation can, therefore, be charged in total directly to the work on which this equipment is being used.

Differences between actual costs of use and



BOARD OF SUPERVISORS.

ROAD COMMISSIONER.

One Chauffeur for Personal Service.

DEPARTMENT OF GENERAL ADMINISTRATION.

1 Executive Secretary.
1 Chief Bookkeeper.
1 Demand Clerk.
2 Bookkeepers.
1 Shipping Clerk.
3 Clerks.
1 Stenographer to Commissioner.
2 Stenographers.
1 Stationery Clerk.
1 Office Boy.

DEPARTMENT OF CONSTRUCTION.

Superintendent of Construction.
One Chauffeur for Personal Service.

1 Office Engineer.
4 Surveyors.
4 Instrumentmen.
4 Rodmen.
3 Senior Civil Engineering Draftsmen.
3 Junior Civil Engineering Draftsmen

3 Senior Resident Engineers.
7 Junior Resident Engineers.
— Emergency Resident Engineers (as needed).

DEPARTMENT OF MAINTENANCE.

Superintendent of Maintenance.
One Chauffeur for Personal Service.

1 Superintendent of Paved Ways.
1 Foreman Paved Ways.
1 Machinist Roller Foreman.
1 Chief Road Oiler.
3 District Foremen.
— Oil Truck Drivers.
— Motor Truck Drivers.
— Roller Engineers.
— Road Oil Men.
— Laborers (as needed)
— Teams (as needed).

36 Road Foremen.
Laborers (as needed).
— Teams (as needed).

1 Highway Engineer.
2 Assistant Highway Engineers.
2 Rodmen.

1 Bridge Engineer.
1 Senior Struc. Bridge Draftsman.
1 Junior Struc. Bridge Draftsman.
— Pile Driver Foreman.
— Pile Driver Engineers.
— Carpenters.
— Concrete Foreman
— Form Builders.

Quarries.

1 Quarry Clerk
1 Commissary Clerk.
1 Engineer Electrician.
2 Steam Shovel Engineers.
2 Cranemen.
2 Steam Shovel Foremen.
2 Quarry Foremen.
2 Locomotive Engineers.
2 Motormen and Oilers (Crushers)
1 Dryer Motorman.
1 Chief Cook.
1 Night Cook.
1 Waiter.
1 Blacksmith.
1 Carpenter.
— Laborers (as needed).
— Teams (as needed).

Fig. 2. Chart Showing Organization of Road Department in Los Angeles County, California.

arbitrary charges for use of both these classes of equipment should be pro-rated monthly or quarterly over the functional work on which such equipment may be used. The arbitrary rate per hour should be based on aggregate hours of probable productive use per annum.

Charges for consumption of Class "C" equipment, together with indirect charges for the care, handling and repair of miscellaneous supplies, small tools and equipment, should be pro-rated over the functional work on the basis of total prime cost.

The purpose of establishing an arbitrary rate or price for the use of material or equipment, is, first, to allocate correctly and automatically as large a proportion as possible of the indirect charges, leaving a minimum for distribution by pro-rating; second and most important, to eliminate by this means all or nearly all of the variable elements effecting the comparison of unit costs, thereby promoting accuracy in the determination of relative efficiency.

Plant operation, where it results in productive output, has to be treated as a class of functional work, unless the plant output can be considered in the nature of a manufactured product used on functional work, in which case it should be treated as "production stock." In the event that the plant has no output of a productive nature, charges thereto should be treated as a burden affecting a part or all of the functional work as circumstances may require.

Rent, insurance and interest, heat, light, power and repairs and replacement of plant structure, where the latter does not constitute betterment to the extent of added valuation, should be included in the cost of plant operation. In case replacement will result in betterment, the cost of such should not be charged to operation but to investment account. But where the maintenance does not fully cover the depreciation, the difference should be charged to operation as depreciation.

DISCIPLINE.

In order to promote discipline and the proper performance of work, it is suggested to issue a set of rules and regulations covering conduct and performance, to be put in effect as soon as possible. There should be established a record of time and gang assignment in card index form, used in conjunction with the issuance of written orders covering the assignment and transfer of men. In this way the card record in the office indicates exactly the current disposition of forces in the field, together with a complete history of the service assignment of each individual. The written orders are made in duplicate. The original is given to the individual affected, and the duplicate sent to the office. The individual's card is at once transferred to the proper place in the card record. In this manner it is possible to check at all times the actual arrangement of force as shown by the time sheets against the disposition of the force as shown by the card record. Discrepancies or omissions in the time reports are at once apparent. By consulting this record of assignment of field forces, transfers to meet emergent conditions may be quickly and intelligently handled at the headquarters office. To control the performance of vehicles while absent from their foremen, a system of tickets should be established whereon is recorded, among other things, the time and point of arrival and departure. The use of such tickets provides a means for checking the excess time taken in transportation service, and makes it possible to [illegible] control over all vehicular service.

[illegible]

To facilitate the efficient management of road forces and to record the current conditions and progress of work, a set of maps should be prepared, each of which designed to cover one of the territorial sections. These maps should be of a scale sufficiently large, and, if possible, be covered with celluloid, to allow the entering of memoranda which can be erased from them. The territorial section thus mapped can be sub-divided by lines into a number of smaller areas or divisions, and each of these again in blocks. By numbering the sections, the division and the block, a complete description of any road area will be provided. On these maps should be indicated the existing roads and highways, bridges and culverts, together with the character of improvement and similar general information. These maps can be used in the following way:

Reports of current conditions as made by inspectors to be indicated in a general way by means of symbols; the existence of such symbols on the map indicates to the one using it that certain work is to be performed at the points so marked, thus permitting the determination of the points to which gangs may best be assigned for economic work performance. As soon as the work has been completed, the symbol indicating a defective condition is erased. Since these maps show the complete road system, they will be constantly used for reference. A statistical card record of road divisions and the road units should be kept, giving such initial statistical information as necessary, and on which notice is kept of the work performed, giving in that manner a historical record of the maintenance of such road units and recording dates of inspection. A statistical card record of all bridges should be kept, showing in like manner all historical data, including dates of inspection.

STOCK AND EQUIPMENT.

Where such cannot be combined with central store-rooms, each road foreman should have under his supervision a stock and equipment yard and a central control record of these yards should be kept. The foreman is held responsible for the equipment recorded as in his care. Transfer of material and equipment from one foreman to another must be only by requisition card and the record so designed as to take care of material, new stock and equipment either received or issued. This record will serve as a perpetual inventory of all road implements, and in addition the road foremen should be required, at regular periods, to make inventories on special blanks, in order to minimize the chance of loss through carelessness.

The control over stores should be kept on the basis of a card index system, providing for a perpetual permanent inventory. The aim of these costs and efficiency records are: (1) To tie functional operations into the bookkeeping records. (2) To show the standard cost of each functional operation. (3) To show the efficiency of each functional operation as to cost, time and quality.

SUMMARY.

Although system is the prime factor of efficiency, there is always danger of over-system when it does not go hand in hand with the establishment of correct standards of personal service, material and equipment service. To set up these standards requires expert practical and technical knowledge of the field work.

Summarizing, it may be said that the whole system of cost accounting as proposed is, in fact, only an extended requisition system, applied to the use of service (labor), material and equipment, set up in card system in such manner that the same card may be used for the purpose of proper accounting as well as to support the standard of efficiency. These fundamental parts of the system are records made at the time of issue; they are reliable, immediate, adequate and permanent and therefore fulfill all requirements of efficiency.

Inability to predetermine and accommodate all of the detail procedure necessary for the proper accomplishment of the work of establishing such a system makes the reorganization work always more or less slow, but the hasty application of new plans to existing conditions may very often result in unsatisfactory conditions.

Field records for a compilation of data under actual conditions will have to be studied to draw up preliminary forms and procedure, and tests will have to determine their acceptability, and may show the possibility of having omitted some features of work performance which was not readily foreseen. Not only cooperation but active and energetic efforts of the division heads is absolutely necessary to develop it to a satisfactory basis of efficiency, and even then it will be necessary to provide for the actual performance of the work by some one who understands the theories on which the system is based, and who may be expected to discover and correct more quickly its faults, after having been fully familiarized with the actual performance.

Rate of Wear and Strength of Granite Paving Blocks.

The following notes from the report of the Bureau of Highways, borough of Manhattan, New York, present some facts illustrating the rate of wear on granite block pavements and the relative desirability of such wear.

Formerly granite paving blocks in use in this and other American cities were admittedly unsatisfactory. There was little attempt at cutting to accurate dimensions and securing uniform width in the different courses. They were laid with wide joints filled with coarse gravel and tar on a sand foundation, so that traffic speedily caused depressions and the attrition from horses' hoofs and steel tires caused the edges to wear and the heads to become rounded and polished. This was also true of the socalled belgian block, in many cases of trap rock of hardness equal to granite. The blocks were too long, rendering them liable to crack or settle unevenly under eccentric loading.

The first improvement in this type of pavement was the introduction of concrete foundation. This, in a degree, obviated the depressions in the surface due to unequal settlement, but made no improvement in the other conditions.

It is now universally admitted that granite block for pavement should be accurately cut to standard dimensions and laid with close joints. There are, however, two distinct points of view in regard to the physical composition of the blocks used. One of these is that all deterioration in stone pavement is due to wear consequent on lack of sufficient crushing strength and hardness, and that only the hardest granites can stand the wear of traffic under present conditions.

The other point of view is exemplified in the following statement of Mr. McClure, city engineer of Worcester, Mass., which is of interest, as that city has made extensive use of granite for paving purposes and its streets are in excellent condition:

> I attribute much of our success in the use of granite blocks to the fact that we have always used the New Hampshire granite, the greater part of the blocks coming from the Marlboro quarries, which is soft enough to wear without polish and hence, with a slight degree of slipperiness.

A similar point is also made in the following statement quoted from a paper written recently by Col. R. W. Crompton, M. Inst. C. E., Great Britain, and published by the Institution of Automobile Engineers:

> Many people, including a considerable proportion of the road surveyors, have, up to quite recently, looked on the question of the wear of road surfaces rather superficially, that is to say, they have always considered that surface wear was the most important matter, and have selected for their macadamized roads the stones from those rocks which offer the greatest resistance to attrition. In this, however, they are wrong.

It is proposed to define proper granite as that which would show a minimum limit under the crushing test of 30,000 pounds to the square inch and have a French coefficient of wear not less than 15. In support of the claim that these qualities are essential it is asserted that all deterioration in stone block pavement is due to wear consequent on lack of sufficient crushing strength and hardness, and that no other pavement than one composed of the hardest granites can stand the wear of traffic under modern conditions. The imperfect condition of many of the older granite pavements in New York is asserted to be due to lack of hardness, which has caused splitting of the blocks and rounding of the edges, and granite has been tested for a compressive strength of over 30,000 pounds to the square inch. The latter claim is literally true.

It may be stated, however, that rounded blocks are not an evidence of lack of hardness, but rather the contrary. Many of the older pavements where the blocks are in the poorest condition, with rounded tops and extremely polished surfaces, are known to have been laid with hard material, the constant friction of horses' hoofs and steel tires on the edges of the blocks, combined with the wide sand joints, producing the above result. Under similar conditions a soft block would wear on the top as well as on the edges, as can be observed in the Medina sandstone pavements of Rochester and Buffalo. The same rounded wear can be observed on granite, steel or cast-iron stair treads where the only attrition is produced by leather shoes. It is impossible to prove that old cracked and badly worn blocks are defective due to any inherent quality. In many cases where such conditions exist they are known to have been due to fires made on the pavements.

It has been claimed that whitened surfaces on the tops of the blocks caused by the impact of horses' hoofs and steel tires show an improper amount of wear and lack of resistance to it. Nevertheless such a condition has been most apparent on some of the streets recently repaved with granite of a hardness meeting in every way the proposed specifications. This would naturally be the case with any stone pavement having a hardness less than any wheels coming in contact with it.

TABLE I.—MAXIMUM COMPRESSION PRODUCED BY HEAVIEST TRUCKS USED ON NEW YORK STREETS.

Power.	Kind of load.	Loads in lbs.	Tires. Front.	Tires. Rear.	Tires. Total.	Wt. per inch run.	Wt. of truck.
3-horse	Steel pipe, etc.	16,000	4-in.	4-in.	16	1,000	6,300 Cornell & Underhill.
3-horse	Steel pipe, etc.	20,000	4-in.	4-in.	16	1,250	7,000 Crane.
Auto	Steel pipe, etc.	6,000	4-in.	8-in.	24	250	8,000 Crane.
20-horse*	Steel, etc.	166,000	14-in.	14-in.	56	2,964	About 12 tons.
3-horse†	Steel, etc.	35,000	6-in.	6-in.	24	1,458	8,300.

*This truck, owned by Mead Bros., is known as the "Whale" and was built to carry 100 tons. It is so loaded that the weight is evenly distributed to the four wheels.

With 100 tons load the weight per inch run of tire is 3,571 lbs. The greatest weight so far carried is 83 tons.

†This is the heaviest 3-horse platform truck made by James Shephard & Sons.

Wheels carrying the loads given in Table I have invariably several longitudinal inches of contact with pavement. Assuming the load to be concentrated on one inch of longitudinal run, this maximum load per square inch of pavement would be 3,571 pounds.

The tabulation in Table II was made by Mr D. B. Goodsell, assistant engineer in the bureau of highways.

TABLE II.—ANNUAL RATES OF WEAR UNDER KNOWN WEIGHTS OF TRAFFIC. (Goodsell.)

Name	Kind of pavement.	Tons in 24 hours.	Width in feet.	Tons per foot width.	No. of vehicles in 24 hours.	Annual wear, in inches.	Rate wear per 100 tons.
Brooklyn Bridge	Granite	4,172	8.00	521	1,786	2.500	.0190
Queensboro Bridge	Wood	8,752	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
Washington Bridge	Wood	1,071	16.08	270	[illegible]	[illegible]	[illegible]
Metropolitan Avenue Bridge	Wood	3,665	20.00	[illegible]	1,142	[illegible]	[illegible]
Meeker Avenue Bridge	Wood	1,019	[illegible]	[illegible]	861	[illegible]	[illegible]
Twenty London streets—"Modern Road Construction"	Wood			183		—	[illegible]
Minneapolis experimental pavement	Wood		0.00	111 12 hr.	13,736	[illegible]	[illegible]

Concrete Poles Withstand Storm.—The recent severe storm which raged over New York City and vicinity provided a thorough storm test of the reinforced concrete telegraph poles, of which the Pennsylvania Railroad Co. have erected a large number in this section. The wind, which reached a velocity of 70 to 80 miles an hour, together with a heavy burden of ice and snow, put the telegraph line out of commission, broke down many of the wooden poles, and some of the wooden cross-arms on the concrete poles. In no case however did a reinforced concrete pole collapse.

Snow Removal by the Public Service Railway Company Between Jersey City and Camdem, N. J., in 1914.

The following article from a paper presented before the Philadelphia Snow Removal Conference by Mr. Martin Schreiber, maintenance of way engineer for the company, describes the method of organization and the equipment and methods of conducting work in removing snow used by the Public Service Railway Co. of New Jersey.

The company operates some 865 miles of single track, through 140 municipalities, having a population of about 2,000,000. The territory is divided into six divisions, extending from Jersey City to Camden, N. J., with an approximate area of 1,100 sq. miles. The procedure in each division varies somewhat on account of local conditions, but generally the principles are the same, as the property is operated from a central office through a departmental organization.

ORGANIZATION FOR SNOW REMOVAL.

No special supervisory organization is required in handling snow, only the regular force of employes is enlarged. Early in the fall the various departmental heads and their assistants, including the superintendents of transportation, maintenance of way, distribution and equipment meet and lay out a coordinate plan of action. The transportation department, through the division superintendents and supervisors at the various carhouses assign certain crews to operate the snow fighting apparatus, such as plows, sweepers and levelers. The names of the crews, with their addresses, are posted in each carhouse in advance so that they may be called on at a moment's notice.

Likewise, the equipment department, through the division master mechanics and carhouse foremen, select the proper mechanical men to accompany the snow fighting equipment and others to look after repairs continuously throughout the period of the snow storm.

The maintenance of way department, through the division roadmasters and section foremen, also records the addresses of a large number of emergency men and teams [illegible] can be quickly assembled if anticipated [illegible] ted.

At the preliminary meeting [illegible] department the [illegible] [illegible] [illegible] [illegible] such as [illegible] curves and [illegible] so that the track foremen may [illegible] tion at the most important places and [illegible] [illegible] [illegible] order.

The distribution department [illegible] [illegible] all the plans [illegible] [illegible] [illegible] how to increase the [illegible] out feeders, in order to take care of the heavy drain on the distribution system that is caused by the operation of the plows and sweepers. Besides, they arrange to have extra emergency crews at certain locations to take care of line breaks.

INSPECTION.

Along with the definite organization scheme, the snow equipment is carefully inspected and all necessary repairs made to the snow plows, sweepers and levelers during the summer, so that the entire apparatus is in first-class condition.

The equipment consists of 76 snow sweepers, 21 shear plows, 2 rotary plows and 7 work cars equipped with levelers. A careful inspection is made of the railway strip to make sure that no paving stones, crossing planks or other obstructions exist, which would interfere with the operation of the plows and sweepers. Sewer manholes and dumps are selected which will be the most economical and satisfactory to the city and also means of access to these places by the trucks are also planned.

All of the plows and sweepers which have been assigned to particular trackage are actually operated on these lines on trial trips, and all troubles which are possible to eliminate are promptly remedied.

METHOD OF CONDUCTING WORK.

As soon as a storm sets in, all of the departments marshal their crews and extra men at the various yards and carhouses in each division. Ordinarily one trackman inspects ten miles of track. During a storm at least six men are kept busy, cleaning switches and special work on a section of this length. The crews for the plows and sweepers are kept on waiting orders at full pay, so that no time will be lost in getting into action. As soon as the snow reaches about 2 ins. the sweepers are brought out and operated continuously during the entire storm. Telephone communications are established and the transportation department assigns a man to keep hourly records of the location of the regular operation on the 111 lines of the system. The record of the last snow storm consists of some 72 typewritten pages, which form interesting data for review.

METHOD OF KEEPING HOURLY REPORTS OF THE OPERATING CONDITIONS.

March 1, 1914.

Essex Division—

At 3:50 p. m. a pole was across both tracks of the Springfield Line, causing no power west of Tenth street on the above line.

At 4:45 p. m. the South Orange Line is reported blocked, due to wires down west of the carhouse.

Hudson Division—

At 6 p. m., Greenville Line—sweeper off track at Greenville cemetery.

Essex Division—

At 6 p. m. car on Black Road Line jumped the track between the bridges, blocking both rails.

At 8 p. m. fire truck stalled on track at South Orange avenue and Twelfth street.

Fourteen cars of Bloomfield Line stalled in Verona because power is shut off.

9 p. m.—Plank Road Line maintaining service from carhouse and shops only. About one half of the cars are stalled in Jersey City, and the headways are being kept up as good as possible under existing conditions.

9:[illegible] p. m.—Springfield sweeper No. 5131 pulled off road on account of controller blowing up.

Bergen sweeper No. [illegible]110 reported with broken [illegible].

12 midnight—Two horses killed by broken electric wire on Twelfth avenue just south of [illegible] avenue.

Passaic Division—

[illegible] on March 1, 1914—The Police Department of the City of Passaic ordered the [illegible] of the Public Service Railway.

Snow collected on the sides of the tracks, through the operation of plows and sweepers [illegible] is carried away from the [illegible] [illegible] means of [illegible] [illegible] of the sweepers [illegible]. Depending on the amount of the [illegible] there [illegible] assigned to each plow or sweeper for the purpose of cleaning out snow [illegible] hand shovels wherever drifts [illegible]. The rotary plows are reserved for [illegible] heavy drifts which occur more frequently in outlying districts or on suburban lines.

When the snow begins to collect faster than it can be handled with the snow removing apparatus, the outlying lines are abandoned first and the apparatus concentrated on the city service. During the last storms the company had a number of cars which were snowed in for 24 hours. This was caused by broken down telegraph and telephone lines crossing lighting and trolley circuits, introducing a great source of danger so that the power had to be shut off. With the power off the snow was quickly packed around the cars and they could not be moved until actually dug out by hand labor.

REMOVING THE SNOW.

As soon as the storm is over, or in fact as soon as the snow is piled in any quantities along the side of the tracks, the work of removing it is started. The largest part is hauled away by teams and dumped into sewers, rivers or meadows; in one city a great quantity was dumped into a canal. Considerable progress has been made in hauling away snow, in some cities, by avoiding conflicting organizations working on the same street at the same time. Definite schedules are mapped out so that the railway company removed all the snow on certain highways between building lines, and the city all the snow on other streets. Some municipalities remove all the snow themselves and the railway company is billed for their portion of the cost on streets where tracks are constructed. One of the customs of the company is to follow up the men on snow removal with special provision cars, so that the men are supplied with hot coffee and sandwiches, all of which has been greatly appreciated by the employes.

SUGGESTED IMPROVEMENTS.

It is doubtful, under present conditions, if it is practical for the railway company to make many more improvements for cleaning track and roadway of snow. Of course there is always room for improvements by strict planning, inspection and supervision of the work. One of the most unfortunate conditions surrounding the handling of snow by the railway company, or municipality, is the fact that the apparatus is idle such a large portion of its existence. The possibility of the mechanical equipment being out of commission so long a time will always be the cause of strong argument against the development of any special machine for melting or removing snow

It appears that about the only way to efficiently remove snow is through further cooperation of the railway company and municipality—for example, stations and dumps could be established and the railway company contract with the municipality for hauling all the garbage and refuse. So, in the winter, in case it was required to haul snow, all the equipment for garbage, including the stations and dumps, could be pressed into service for disposal of the snow, without any extra investment expense.

Another plan would be to design streets and roadways with special gutters equipped with flushing apparatus, which could be used for cleaning and draining streets as well as getting rid of the snow.

Notes on Roads and Streets.

Roemac.—The following conclusions are made in an article by Robt C. Muir in the Canadian Engineer.

Roemac is a silicate-saccharate-carbonate, the foundation of which is silicate of soda, sugar and carbonate of lime. The sugar is not used for adhesive purposes, but to assist the chemical action of the silicate on the carbonate, and secondly to assist the resistance of the matrix to frost.

A matrix is formed of the mixture in the proportion of ⅓ cu. yd. of crushed limestone to 5 gal. of Roemac solution with an addition of similar proportion of water. This is mixed similar to the making of concrete, the limestone crushed to ¼-in gage and dust. This matrix is applied to existing road surface to a thickness of 1 in. for a finished 3-in. coat. The stone is then spread on and rolled until the matrix comes to surface. All superfluous material is swept off and surface left clean and hard.

The advantages of Roemac are:

(1) Repairable in almost any kind of weather, except hard frost.

(2) Cheaper than bituminous macadam.

(3) No yearly surface painting required.

(4) Dust not so obnoxious as from tar.

(5) Good foothold for horses, not so slippery as tar macadam.

(6) When laid, frost has not injurious effect on it as upon other road surfaces.

The defects of Roemac are:

(1) Dustiness in summer, though after superfluous limestone has been washed off dust is greatly reduced.

(2) Hard, metallic ring.

It is claimed that the life of Roemoc road is 3 times that of an ordinary water-bound macadam road. The cost of Roemac was 52 cents per square yard.

The author's experience has been that with roads laid upon a clay subsoil, imperfect drainage and under a medium heavy traffic tar in any shape or form soon gave out. After careful consideration he came to the conclusion that Roemac, though not ideal in every way, was well suited to roads under these conditions.

Cost of Equipping a Convict Force.—In a paper before the American Road Builders' Association Mr. W. E. Atkinson, state highway engineer of Louisiana, states that an approximate estimate of the cost of equipping and running a convict camp for road purposes, based upon a force of laborers numbering 50, would be as follows:

Item.	Cost.
10 teams, at $500.00	$5,000.00
10 road slips, at $7.00	70.00
1 road grading machine	200.00
1 road roller	2,000.00
2 road drags, at $20.00	40.00
1 blacksmith outfit	50.00
All necessary axes, picks, shovels, etc.	100.00
Tents for living quarters, dining room, stables, etc.	500.00
Kitchen outfit	50.00
Lumber for cell-house, etc.	500.00
Incidentals	500.00
Total	$9,010.00

A camp of 50 men should be supervised by the following superiors:

1 captain, at $75.00 per month	$ 75.00
3 foremen, at $40.00 per month	120.00
5 guards, at $30.00 per month	150.00
Total	$345.00

From actual experience, it has been ascertained that the cost of maintaining and operating one convict per year in Louisiana is about $170, or between 45 and 50 cts. per day.

Cost of Dust Laying with Dustoline.—The cost of dust laying in New Bedford, Mass., as given by Geo. Hye, city engineer, is shown in Table I. The table covers operations extending over the season of 1913.

TABLE I. — THE COST OF DUST LAYING WITH DUSTOLINE ON 109 MILES OF STREETS IN NEW BEDFORD, MASS., DURING 1913.

Item.	Cost per gal. applied.	Cost per sq. yd., one coat.	Total cost.
Material	$0.0857	$0.0163	$20,598.17
Labor	.0040	.0008	960.96
Totals	$0.0897	$0.0171	$21,559.53

Total material used, 240,421 gals.

Material used per sq. yd., one coat, 0.19 gals.

	Sq. yds.	Miles.
First coat applied to	1,003,336	80.22
Second coat applied to	296,920	23.73
Third coat applied to	62,322	4.67
Total	1,262,578	108.62

Road Damage by Motor Buses.—The following note is taken from a recent consular report:

The heavy wear on roads, particularly macadam roads, from motor-bus traffic, is causing not a little concern to the British authorities responsible for road maintenance. A couple of years ago motor-buses were started by a private company under a temporary license in Birmingham, and about a year ago the city put on a few motor-buses as feeders to the city street car lines. The private motor-buses are shortly to be taken over by the city. The destruction of roads by the use of these motor-buses has been very noticeable, and authorites are not only considering methods to lessen this destruction, but in the accountings of the various city municipal undertakings the question of which department should bear the extra cost of road repairs due to motor-buses has still to be decided. Where motor-buses have been run by private companies, as in a number of cities throughout the Kingdom, much objection has been raised because they were said to contribute sums in taxes that were relatively unimportant compared to the wear they caused on roads.

Street Accidents in England.—A recent consular report states that highway accidents have increased so rapidly in the United Kingdom since the very extensive introduction of mechanically propelled vehicles that in the 12 months ended Dec. 31, 1913, the number of those killed on the highways in town and country reached the large total of 2,099, while 42,544 were injured, which was an average of nearly six killed and of more than 122 cases of injury for each day of the year.

TABLE I.—NUMBER OF ACCIDENTS IN FIVE YEARS.

Years.	Killed.	Injured.	Total.
1909	1,151	16,872	18,023
1910	1,327	29,750	31,077
1911	1,557	33,653	35,210
1912	1,764	36,833	38,597
1913	2,099	42,644	44,643

TABLE II.—DISTRIBUTION OF ACCIDENTS AMONG DIFFERENT CLASSES OF VEHICLES IN 1913.

Classes.	Fatal.	Non-fatal.
Motor cars, motor vans, etc.	971	18,497
Electric street cars	178	6,708
Motor omnibuses	226	3,722
Horse omnibuses	7	66
Horse-drawn street cars	...	40
Other horse vehicles	717	13,511
Total	2,099	42,544

TABLE III. — FATAL ACCIDENTS (1910-13) AND CHARACTER OF VEHICLE CAUSING ACCIDENT.

Classes.	1910.	1911.	1912.	1913.
Motor cars, motor vans, etc.	478	633	751	971
Motor omnibuses	82	117	199	226
Electric street cars	112	123	135	178
Horse omnibuses	15	10	7	7
Horse-drawn street cars	1	4	...	...
Other horsed vehicles	639	670	672	717

In the city of London proper, which embraces the mile square in what may be called the financial section of London, 17 persons were killed and 1,210 injured in 1913, while in the metropolitan police district 579 were killed and 18,365 injured. In the whole of the London area, which includes both the city proper and the metropolitan police district, 190 of the deaths resulting from street accidents were caused by motor omnibuses, 187 by other motor vehicles, 58 by electric tramcars, and 161 by horse-drawn vehicles other than street cars or omnibuses.

Injuries to 7,174 persons were inflicted by motor cars, motor vans, etc., while 3,683 accidents were caused by motor omnibuses, 3,053 by electric street cars, 20 by horse omnibuses, 25 by horse-drawn street cars, and 5,720 by other horse-drawn vehicles.

TABLE IV.—ACCIDENTS IN METROPOLITAN POLICE DISTRICT AND LONDON PROPER.

	Metropolitan police district.		City of London.	
Years.	Fatal.	Non-fatal.	Fatal.	Non-fatal.
1909	287	11,731	16	1,354
1910	348	12,493	19	1,363
1911	385	13,849	25	1,405
1912	514	15,291	24	1,360
1913	579	18,365	17	1,210

Liverpool came next to London with 33 deaths and 1,765 accidents, followed by Birmingham with 44 deaths and 1,546 accidents, by Manchester with 41 deaths and 1,085 accidents, and Glasgow with 40 deaths and 845 accidents.

Engineering and Contracting

Devoted to the Economics of Civil Engineering Design
and to Methods and Cost of Construction

Volume XLI. CHICAGO, ILL., MAY 13, 1914. Number 19.

Concrete Road Construction in Milwaukee County, Wisconsin.

The concrete road construction work in Milwaukee County, Wisconsin, during the past eason has in a large measure demonstrated wo points: First, that a uniform density of he sub-grade is an important factor in eliminating the possibility of cracks in the surace; and second, that the average cost of constructing a first-class concrete surface on the average country road is approximately $1.50 per square yard.

At the present writing four small transverse cracks exist in the 22 miles of concrete surfaced roads constructed in this county during he past season. In comparison with the results of the first season in this and other sections this result shows that a distinct improvement in methods has been obtained.

There is no question but that the proper preparation of the sub-grade has in former years been neglected. This is coming to be realized by engineers, as the following extract from the report of the committee on recommended practice at the National Conference on Concrete Road Building, held in Chicago, Feb. 12-14, 1914, shows:

> Excavation or Treatment of Existing Roadway.—The fundamental requirement of the subgrade is that it should at all times be of uniform density, so that it will not settle unevenly and cause cracks in the surface of the pavement, and no part of the work is more worthy of intelligent care and painstaking labor than the preparation of the sub-grade; the slight additional cost necessary to insure good results is abundantly justifiable. When the pavement is constructed on virgin soil, care should be taken to remove all spots so as to insure a uniform density; and if constructed on an old roadbed even greater care must be taken in preparing the sub-grade, which is likely to be more compact in the center than at the sides, and consequently there is more danger that the pavement will settle unevenly, causing cracks.
>
> Drainage.—The drainage of the roadbed of a pavement is of vital importance. If the subgrade is not well drained there is danger of unequal settlement after the pavement is laid, causing longitudinal cracks; there is also a possibility that the frost may lift the edges of the pavement and likewise produce cracks. The character of drainage necessary will depend on local conditions. It is recommended that in general proper drainage may be secured through lateral ditches. If underground water is present this may be drained through the use of drain tile laid at a suitable depth on either side of the roadway, or on one side with cross drains from the other side. The top of the tile should not be less than 3 ft. below the sub-grade.

We believe, however, that while the density must be uniform that the greater the density the more secure the foundation will be and that this is especially true with regard to the danger of frost action.

The careful and conscientious testing of all material used for aggregate is essential in securing a surface which will wear uniformly and in Milwaukee County such testing was conducted consistently. In this connection it is well to remember that where great precautions are necessary to secure uniformity of materials and methods the cost of construction is apt to be materially increased. Moreover, on small jobs these costs will be proportionately higher than on large construction jobs such as were carried on in this country.

A first-class concrete road surface cannot be constructed at the low first cost which was thought possible sometime ago. It is seldom that the average haul of material on a country road falls below 1½ miles and this fact combined with many precautions it is necessary to observe both in the preparation of the subgrade and the selection and inspection of methods and materials of construction will have such influence that the first cost will be materially increased over that given in many published cost data of work done in the past.

There is no doubt that concrete road surfaces have a definite place in country road construction but they are not imperishable nor are they adapted to all localities.

Methods and Costs of Reconstructing Buildings.

The uncertainties of reconstruction work render it difficult to make a close estimate of its cost. It is often found that incomplete records of the details of the existing plant have been kept, due to the fact that changes in the original plans during construction were not recorded. Such work is further complicated when it is necessary to continue the partial operation of the old plant during its demolition and replacement. The uncertainties involved in such work often make it inadvisable to let the contract for a fixed sum, unless provisions are made for the numerous extras which are always a fruitful source of disagreements. Reconstruction work in which it is necessary to continue partial operations can usually best be done by the company's forces, if they are equipped to handle the work, or under some form of contract involving actual cost plus a fixed sum or percentage.

It seldom happens that it is advisable to proceed with the design and construction of building foundations without making investigations of subsoil conditions and ascertaining the location of drains, sewers, etc. Such investigations will often result in a large saving in the cost of the completed structure.

In the buildings section of this issue we are publishing an article on the methods and costs of reconstructing an old bay of the car shops of a street railway and light company and of constructing a new bay adjacent to the old one. In this case it was necessary to continue work in the old bay during its reconstruction, thus making it necessary to build the structure in sections. During the excavation of the basement of the new bay a 3-ft. sewer was discovered near the surface. As this sewer was directly under a line of columns, for the entire length of the building, it was necessary to lower it by means of jacks. The excavation work for the new bay was also complicated, due to the existence of a large number of electric light poles, which were braced and lowered as the excavation proceeded and were finally removed from the site without interfering with the operation of the electric light plant.

The walls of the old bay were badly out of line and dry rot had destroyed a large part of the section of the wooden stringers. It was necessary to underpin the walls and carry the new footings several feet below the old ones. By adjusting carefully the jacks which were used in underpinning the walls it was possible to bring the old walls back to their vertical alignment. The necessity of using care in waterproofing a thin concrete roof slab is emphasized by the unsatisfactory results obtained in this building.

As construction work is often done by men not entirely familiar with such work (as in this case), it is instructive to note the unit costs under the stated conditions. The completeness of the cost data and the full statement of conditions under which the work was done are noteworthy.

Organization and Equipment Data as An Aid To the Road Contractor.

Road contracting calls for a large initial outlay for equipment. Money paid out for labor constitutes the greater part of the cost of the average road job. Therefore there are no two factors which play more important parts in the success or failure of a road contractor than, first, proper purchasing, balancing and disposing of his equipment and, second, perfecting a smooth running organization in which the labor cost is under constant observation and control.

It is the practice of successful designing engineers, before starting on the design of any important work, to gather all the data available on work of the character in hand. They surround themselves with plans and descriptions which may furnish hints helpful in the preparation of portions of their design. Successful contractors of today follow the same practice in regard to methods, organization and equipment. The data may have been gathered from their own experience—if so, so much the better—or they may be data collected from the experience of other men and preserved or published in the technical press. Regardless of their source, accurate and detailed data have a very definite value as a check and guide to the judgment in planning and estimating road work.

A Comparison of Day Labor and Contract Work Based on Experience at Winnipeg, Canada.

Municipal construction by day labor has been developed to greater economy by the city of Winnipeg, Canada, perhaps, than by any other Canadian city. In the "Contract Record" for May 6, 1911, Mr. J. W. Astley, Engineer of Construction for the city of Winnipeg, discusses the conditions which have brought success and presents general conclusions in respect to day labor and contract construction based on his experience. We quote as follows:

I may state that in forty-five years' engineering experience, I have had, for ten years, the responsibility of handling every class of civic work by day labor. For the last six years I have carried out this work under a system somewhat different from what is usually employed. In this respect I tender against outside contractors, who are furnished city material at the same prices that I am charged. In my tenders I include the following overhead charges; interest, sinking fund and repairs to all machinery, time-keeping, book-keeping, superintendence, and a percentage for operating stores department.

A contractor has to carry his own fixed charges, so that practically the only difference between day labor, as I have to carry it out, and work done by a contractor is that I put up no deposit and the city becomes my security for the due fulfillment of the work. The inspection is supposed to be the same, but it is carried to the extreme in my case, so as to prevent any loophole for complaints. I am expected to carry out my work for my tender prices, and I have done so with very few exceptions. The average yearly value of work for the last six years has been close to $1,000,000.

A detailed statement is submitted at the end of the year and is published, so as to give the general public a chance to see the results of day labor. I am giving a plain statement of facts, as they carry great weight in regard to this question.

The city of Winnipeg owns and operates the following public utilities: unlimited electric power for light and power purposes, water works—domestic and high pressure, gravel pits, stone quarry, crushing plant, railway cars for handling material; one stationary and one railway asphalt plant, storage yards with overhead supply bins, and machine shop equipped with up-to-date appliances; stores department, gasoline trucks, tractors, rollers, sewer machines—in fact everything that makes a contractor's outfit for certain classes of work and a certain amount of work.

The cost of material is as follows: broken stone, delivered, per cubic yard, $175; cement, in stock, per barrel, $2.10; gravel, delivered, per cubic yard, $1.25; asphalt, per ton, $25.00; flux, per ton, $22.50; limestone dust, in stock, per ton, $4.50; crushed granite, per cubic yard, $3.50.

Fixed charges are kept sufficiently high to leave a safe margin on an ordinary season's work. Naturally, if a very big season's work is carried out, more is raised than would cover what was calculated as wear and tear, etc. This extra amount is used to further reduce capital accounts on various plants.

Standard asphalt pavement, 3-in. ballast, 6-in. concrete, 1½-in. binder and 2-in. surface, costs $2.65 a square yard. This includes superintendence and timekeeping, 3 cts.; sinking fund, interest and upkeep of plant, 7 cts. per square yard; maintenance charge, 20 cts. Total 30 cts. per square yard.

Extra fixed charges on all works:

4 per cent on labor for general tools and plant.

3 per cent on cost of stores for operating same.

3 per cent on labor for superintendence and timekeeping.

$15.00 per day for use of large concrete mixers; smaller size, $7.50.

$15.00 per day for steam rollers. $25.00 per day or its equivalent for gasoline tractors.

$25.00 per day for sewer machines.

These charges are exclusive of engineers' and helpers' wages.

The price per square yard for artificial stone sidewalks, of which there were 84 separate contracts last season, was:

For residential streets, 8-in. gravel base, 4-in. concrete and 1-in finish, per square yard	$1.55
For business streets, 8-in. gravel base, 5-in. concrete and 1½-in. finish, per square yard	1.65

Prices per foot for 6-in., 8-in., 10-in., and 12-in. water mains were 27 cts., 30 cts., 35 cts. and 40 cts. respectively. This includes lead, hemp, and connecting to existing mains and hydrants. All water pipe and specials are furnished free by the city.

Prices for 12-in., 15-in. and 18-in. pipe sewers were 65 cts., 75 cts. and 90 cts. These prices include all except cost of pipe.

Concrete Sewers.—The general average of cost of concrete is $8.00 per cubic yard, and excavation 50 cts. per cubic yard I may state that this class of sewer is generally very deep, and our ground is known to be very treacherous, thus preventing the use of certain classes of sewer machinery.

With the aid of these prices and our fixed charges, contractors and city officials in other localities will be able the more readily to adjust any difference in cost of labor and material

It will be seen that by having our own plants, we are well fitted to carry out a large amount of work and thereby keep our machinery working as many days as possible during the working season. At the best, in this climate, six months in the year is the most you can calculate on to make any machinery earn towards its cost of upkeep.

At the same time, it often occurs that tenders are called for more work than can be done in a given time by the machinery we own, and as it would be a poor policy to expend $10,000 to $12,000 for extra machinery simply in cases of this kind, outside contractors, in such instances, are expected to get this extra work. At the same time, however, prices are kept down to a fair basis by the knowledge that the city could handle all the work if they wished to do so by purchasing more machinery.

Another very important fact that must not be lost sight of is that no matter how much or how little work a city may have, they cannot do work outside of their own civic works so as to try to keep both their men and machinery steadily employed. A contractor has this privilege, and can tender on other work as well, thereby giving him a chance to keep a well-organized gang steadily at work. Also the fact must be borne in mind as regards all civic works, that seldom is any endeavor made to order different undertakings so that one could be completed before the next is started. They are all usually ordered together, and must be carried out at once, as they are wanted immediately, for the simple reason that many of these expenditures have been put off as long as it was possible to do so with public safety. The city powers that be cannot be blamed for this, as it is only too often simply a question of finances.

Outside of a city having reasonably up-to-date plants, without having gone to the extreme for reasons given in a previous paragraph, they fail to do as cheap work as a contractor unless a competent man is given full control of men and machinery, and held responsible for the cost of the work. This is as good security as that put up by any municipal contractor, as any man who is fit for the position should never be put there by civic influence. He must stand or fall by past and present record, and any question of whom he shall hire must be cut out. In plain language, cut out ward patronage or quit day labor once and for all.

It may be considered by many that our fixed charges are excessive, but this is one item that is not given sufficient thought by city officials when contemplating the day labor question. Contractors have it drawn to their attention very forcibly when their hands have to go into their pockets for new machinery. My idea on this subject has been formed from many years' use of every class of machinery and I believe that as the city cannot claim or make use of any direct saving in the cost of any work, such saving going to the benefit of the property owners direct, a liberal collection should be made which would leave the cost of the work at a reasonable figure.

By this means, after the assumed life of any machinery is passed, and consequently paid for, it can be scrapped if thought advisable, either on account of the cost of the yearly repairs exceeding one-half the expected revenue, or because it has become out of date. The question of machinery becoming antiquated is a much more important one than many people think. Many classes of machinery are out of date in five years, where the life might be safely placed at ten years. Once the saving on operating on account of any improvement will more than pay for the general upkeep of the older machine, it is time to scrap it, otherwise your outfit will be going further behind before the next change occurs.

It can also be taken as a fact that will bear no question of an argument, that the average of all work will be of a better quality than if done by many so-called municipal contractors. Many of this class of contractor are here today and gone tomorrow, and as very often happens in municipal work, many unforeseen difficulties crop up that cause the work to be more expensive than was assumed, and still cannot be claimed as an extra. A contractor, unless he is overburdened with cash, must slip over these places at the least loss to himself, whereas the city can, and should, afford to expend whatever is necessary, even if by so doing the cost overruns the original tender price.

If it were possible for one or two reliable municipal contractors to be awarded all city work, it would be a different matter. They would then have the same incentive to do this work as a lasting record to themselves. If a town or city decides to adopt the day labor principle, there are many things that they had better leave alone, especially large buildings and certain classes of other work that only come once in a while, and require a different plant than that in general use. This woul mean carrying a large outlay in dead plan besides requiring certain skilled labor that i too high priced to be used on any ordinar work; consequently on these odd jobs yo would only get the left overs in the way c skilled labor, which is dear at any price.

WINNIPEG PROCEDURE.

After tenders are advertised for and bid are received, the tender which is lowest i recommended by the Board of Control to th Council, and if my bid is accepted I am not fied to proceed. You will thus perceive tha the two governing bodies of our municipa system have made themselves responsible t the people for the due fulfillment of the worl both as regards the time limit for completio and the stated cost of the work. Under th circumstances, I consider that I hold a ver responsible position, having the reputation c the two civic governing bodies as well as m own at stake, and must "make good."

I will state that I have never been aske by any members of the Board of Control c City Council to accept any man as forema on these works. The same applies to laborer If this system is carried out, day labor ca be made a success, and the average cost of a work will be cheaper than any contractor ca afford to do it for.

There are exceptional cases when, if a co tractor wishes to keep his gang together fo other work he has in view, he can afford take civic work at cost prices, in which ca if he comes out clear, he has actually made pay, as the loss in wages of making up a ne gang amounts to a considerable sum in do lars, besides the question of efficiency in kee ing a well trained gang ready for the next jo

Winnipeg was the pioneer city to go in the question of building and operating the own municipal asphalt plants and doing a their own asphalt work. The effect of all th class of work being done by the city is ve apparent. We can justly claim a more ev quality of good pavements and cheaper keep in repair than any city on the Americ continent. This is simply owing to a unifor system being carried out in regard to mixi and laying pavements from year to year, a repairs being made as required, not to s the time and convenience of various contra ors as would be the case if repaired under t guarantee system. It would not be possi to lay as even a class of pavements if sever contractors were engaged in the work duri different years, and the figures given in a p vious paragraph will show that no contract could afford to compete and make a fair ma gin of profit.

To make it pay a town or city to go in this class of work, they should be able to s that under ordinary conditions a certa amount would be required every year. there is a rush one year and very little do the next, it means carrying over overhe charges to a second year, which is not a go policy, but if sufficient asphalt pavements a already laid that are past their guarantee riod, and there are others that still ha some years of the guarantee period yet to r these two can always be counted on to ma a certain amount of work for a munici asphalt plant, and will help to carry the ov head charges.'

I am a firm believer in day labor if carri out as a business proposition, but if attempt to be carried out as in many cities south the line and in many of our own Canadi cities, under the old style of city council a ward politics, it cannot be made a success. simply means taking the property owne money and distributing it among the worki class, under the mistaken idea that the city better off because many men who are reside receive employment on civic works, but wh no contractor would hire at any price exc to fill a gap until he can pick up somethi better

I have seen civic works carried out by called day labor on both sides of the bord under practically the above system, and the timated extra cost was at least 30 per cent o what it should be, but as the old saying "the people get what they want and so ha no kick coming."

Buildings

Methods and Costs of Reconstructing the Car Shops of the Macon Railway & Light Co., Macon, Ga.

Reconstruction work often involves greater roblems than develop in the design and erecon of new structures. It is difficult to estimate the cost of reconstruction work, especially where such work must be carried on without interfering seriously with the operaion of the existing plant. The following aricle describes the design features and the methods and costs of reconstructing an old ar shop and of building an adjacent new one or the Macon Railway & Light Co., Macon, Ga., the work being done in 1913. The article is based upon a paper, by Carl H. Fuller, presented before the American Society for the Advancement of Science, although a considrable part of the data, not included in the riginal paper, was obtained by us from Mr. Fuller. It is stated that a new plant, having he same or better facilities, could have been onstructed in a new location at less expense han was required for the work actually done.

OLD CONSTRUCTION.

The old shop consisted a brick building, with a wooden floor and roof, located on a loping lot, one end of which faced the street and the other abutted on the steam plant. A plan showing the layout of the old and new bays is given in Fig. 1. The old shop (and the reconstructed one which it replaced) had a clear width of 48 ft. 3 ins. and a length of about 137 ft. The main floor was reached by hree tracks, which branched off from the barn adder-track, the main car barn being located a short distance south of this shop. The front part contained three pits, accommodating hree cars, and the rear of this floor was occupied by some old wood-working machinery. The rear portion of the basement had been xcavated for a machine shop, and it contained a wheel press, boring mill, lathe and some minor tools. To replace the wheels under a car, the car body was jacked up, the truck was run out from under it, and the wheels were lowered to the machine shop in the basement hrough a trap door by means of a block and ackle. The street end of the building was ntirely open, reliance being placed on this nd a few irregularly placed windows for ight.

In removing the old floorbeams and posts some track stringers, having a 12-ft. span, were found to contain only 1 sq. ins. of sound wood, the remainder being dry rot. The only part of the old building which proved to be worth saving was the brick walls, and these were so far out of plumb in some places that he center of gravity of a 13 in. wall fell 11 ins. outside of the face of the wall.

NEW CONSTRUCTION.

The reconstruction work on the old building consisted of excavating the entire basement area to give a clear headroom of 9 ft., the building of a reinforced concrete floor at the street level; the construction of a 2½-in. reinforced concrete roof on new steel purlins and trusses; and the repair and alignment of the old walls. In addition to this work there was built a new bay of similar construction, 18 ft. wide in the clear by 117 ft. 1 in. long. Each building has large steel sash windows and a steel sash monitor, the sash being operated by a "Drive" operator. Each monitor covers six 16-ft. bays. Both buildings have rolling steel curtains, which extend entirely across their fronts.

The remodeled bay contains a storeroom for the associated companies, which occupies practically the entire basement and is served from the street floor by an electric elevator. It has three tracks which run the entire length of the building, the rails of which are embedded in the concrete floor. The concrete pits in the front part accommodate six short cars or three long ones. A wheel press and a boring machine are arranged across the track in the rear, and such other tools are provided as are needed in a thoroughly equipped machine shop.

The new bay contains three tracks on the street level and four storage tracks in the basement, the latter being reached from the rear by an inclined track from the street. The street level story is equipped with a complete line of wood-working machinery. It is also used as a paint shop.

The shops are electrically lighted, with both incandescent and arc lamps. All machinery is operated by electric motors, several machines being grouped together by means of short counter-shafting and run from one motor. The present stop has complete equipment for building new electric cars and repairing old ones. Work can be carried on at one time on nine cars, without crowding.

DESIGN FEATURES.

New Bay.—At the site of the new and reconstructed bays the ground surface slopes considerably. Figure 2 shows the north elevation of the new bay. The 16-ft. panel at the right of Fig. 2 is the front panel of the reconstructed bay. This drawing indicates the type of construction, and gives the general dimensions for the basement story. It will be noted that the drainage of the roof is effected by means of [illegible]-in. downspouts draining into a vitrified clay pipe sewer, the diameter of which varies from 8 ins. at the front to 12 ins. at the rear.

Figure 3 shows a cross-section of the reconstructed bay and some typical details. This drawing shows the design features of the roof and floor construction and gives the principal dimensions. Figure 3 (a) shows a cross-section of the bay; Fig. 3 (b) shows the north end bearing of the truss; Fig. 3 (c) shows a column footing for two trusses; Fig. 3 (d) is an elevation of a typical track stringer, and Fig. 3 (e) a cross-section of a typical track stringer.

Old Bay.—Figure 4 shows a cross-section through the exterior walls, [illegible] and basement floor of the old (south) bay. The details for the roof construction are not shown as they are similar to those shown in Fig. 3. The truss for this bay is 3 ins. longer than for the new bay, making the length of each of the eight panels 6 ft. 2 ins. instead of 6 ft. 1⅝ ins. Figure 4 (a) show the wall, the columns, and the floor construction; Fig. 4 (b) shows a plan of the intermediate column footings; and Fig. 4 (c) shows a cross-section of a track stringer. New brickwork was used for the walls above the lower chord of the trusses.

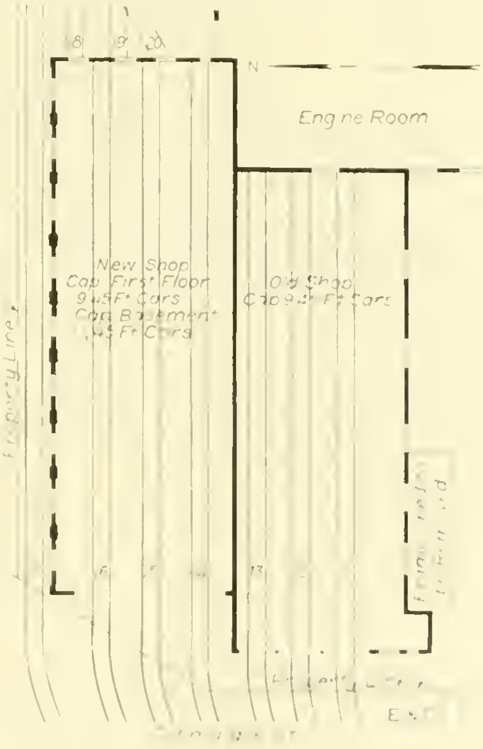


Fig. 1. Plan Showing Lay-Out of Old and New Bay of Car Shop at Macon, Ga.

CONSTRUCTION FEATURES.

As the old equipment was badly run down, and as there were no spare cars, it was necessary for the company to continue the operation of all shop tools and facilities during reconstruction, and also to install new tools. The construction of the new bay and the reconstruction of the old one were in progress at the same time.

Old Bay.—In the old building the excavation work and the construction of the concrete floor and column footing were carried on in sections. The machinery was jacked up, the concrete floor was placed, and the machinery was then lowered into position. The wooden columns were removed, and old trolley poles were substituted as the excavation progressed. About two-thirds of the exterior walls had to be carried down from 2 to 8 ft. below the original footings. This work was also done in sections, old rails (spaced about 12 ins. apart) being placed through the walls and supported on jack screws. The old foundations were then cut away, and the new wall was built up to carry the old construction, the list in the old walls being rectified, so far as possible, by adjusting the jacks at the time of joining the old and new work. This work was successfully completed without the appearance of any cracks, although the walls were laid with a poor quality of lime mortar.

Following this work, one track was taken out for the entire length of the building, the old wooden pit and floor were removed, and the new concrete pit and floor were cast on the top of columns which had previously been constructed in the basement. As soon as a section of the floor was completed and the concrete had set sufficiently, the machinery and car work were moved over to it, and the next section was put in. In most cases the new concrete floor was loaded and was in use within 18 hours after it was cast. The roof construction was also placed one section at a time, the lights, shafting, wiring, etc., being transferred from the old to the new construction as the work progressed.

New Bay.—Only two features of the new construction will be discussed, viz., the excavation of the lot area and the removal of old poles and work occasioned by the discovery of an old 3-ft. sewer.

The depth of excavation varied from 16 ft. in front to 2 ft. in the rear. The lot also contained 38 poles, which carried the transmission wires for the railway and lighting departments. These wires had to be transferred one at a time to a new line of poles which were placed in the alley. It required 60 days to clear away the wires and poles, the excavation being carried on during this time. The poles were guyed to prevent them from upsetting, and were allowed to sink as the excavation progressed. [illegible] the wires carried [illegible] volts, and there was always present an element of danger.

In excavating the lot an old 3-ft. sewer was discovered. This sewer ran parallel to and was [illegible] for the entire length of the building. It had been constructed of brick, paving blocks and old curb stones. When uncovered it was too late to change the plans of the new walls and about one-half of the basement columns had been constructed. As the sewer flowed about half full at all times and as there was no way of diverting it temporarily, the excavation was carried on by flooring over the sewer as the work progressed. The slope of the sewer was about the same as that of the lot, and it had to be lowered from nothing at the lower end to about 8 ft. at the upper end, within the length of the building. A trench was dug under the sewer, and a drop manhole was built on the outside where it entered the front wall. The entire sewer was then transferred to jacks, and the sewer was cut at each end of the building, the water flowing into the trench and filling it. Owing to the use of an insufficient number of jacks, to the conglomerate construction of the sewer, and to the variation in depth to which it had to be lowered it proved to be impossible to lower it intact, and about one-third of it was wrecked in the operation. The broken portion was rebuilt, using bricks laid in cement mortar, the water flowing through the trench during the reconstruction of the sewer.

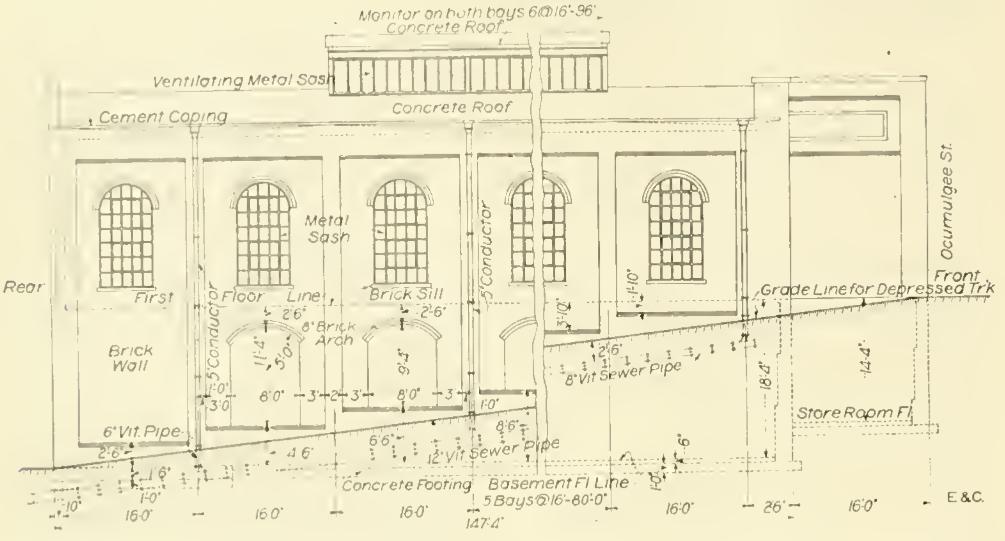
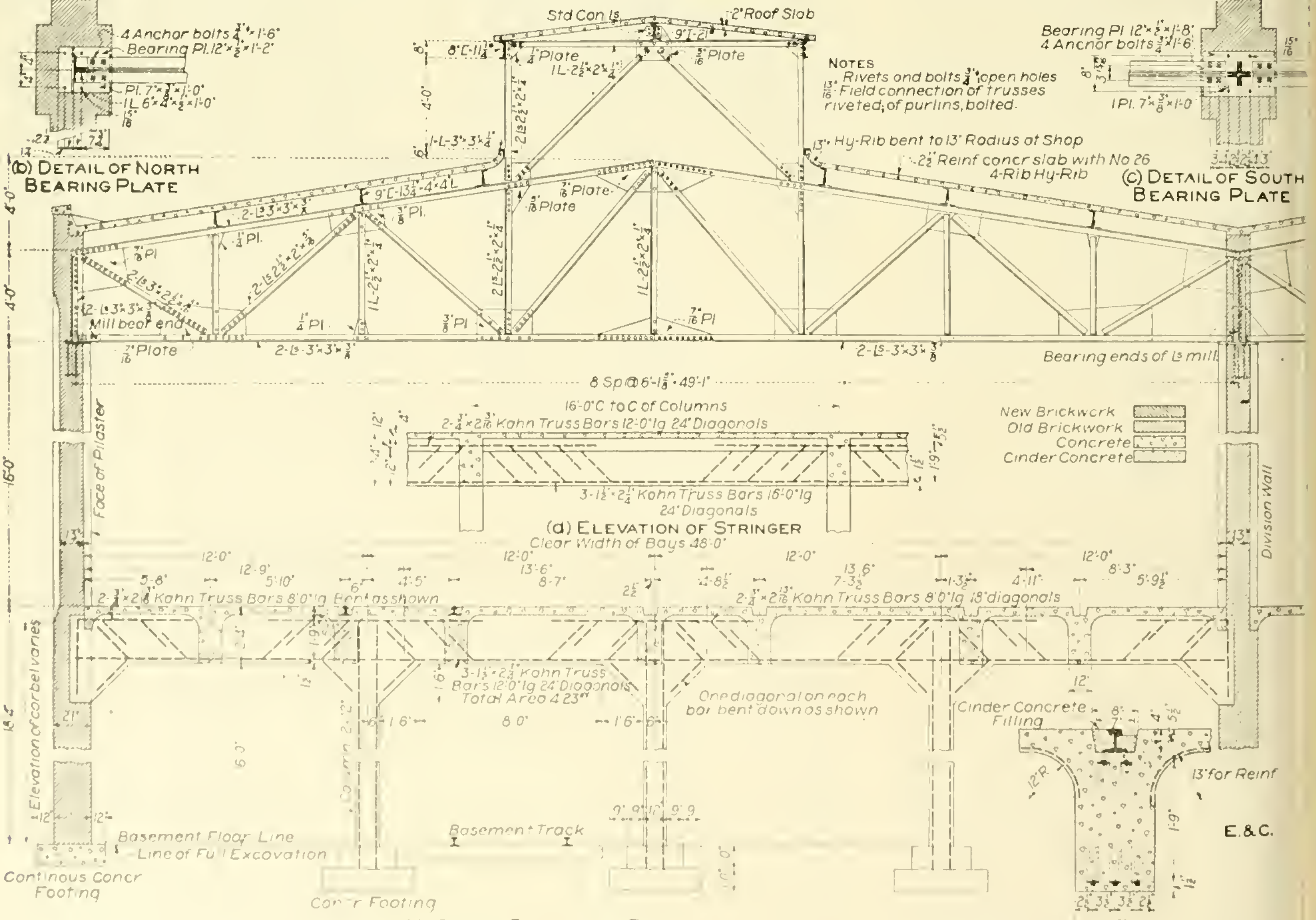


Fig. 2. North Elevation of New Bay of Car Shop Showing Grade of Depressed Tracks and Type of Construction.


Std Con Is.
2" Roof Slab
4 Anchor bolts 3/4"×1'-6"
Bearing Pl. 12"×1/2"×1'-2"
Notes
Rivets and bolts 3/4" open holes 13/16"
Field connection of trusses riveted; of purlins, bolted.
Bearing Pl 12"×1/2"×1'-8"
4 Anchor bolts 3/4"×1'-6"
Hy-Rib bent to 13' Radius at Shop
2 1/2" Reinf concr slab with No 26 4-Rib Hy-Rib
Mill bear end
Bearing ends of Ls mill
8 Sp @ 6'-1 3/8" = 49'-1"
16'-0" C to C of Columns
2-1/4"×2 13/16" Kahn Truss Bars 12'-0" lg 24" Diagonals
3-1 1/2"×2 1/4" Kahn Truss Bars 16'-0" lg 24" Diagonals
New Brickwork
Old Brickwork
Concrete
Cinder Concrete
Face of Pilaster
Division Wall
Clear Width of Bays 48'-0"
Kahn Truss Bars 8'-0" lg Bent as shown
Kahn Truss Bars 8'-0" lg 18" diagonals
3-1 1/2"×2 1/4" Kahn Truss Bars 12'-0" lg 24" Diagonals Total Area 4.23"
One diagonal on each bar bent down as shown
Cinder Concrete Filling
13" for Reinf
Elevation of corbel varies
Basement Floor Line
Line of Full Excavation
Continuous Concr Footing
Basement Track
Concr Footing
E.&C.

(b) Detail of North Bearing Plate

(c) Detail of South Bearing Plate

(d) Elevation of Stringer

(a) Cross-Section of Bay

(e) Cross-Section of Girder

Fig. 3. Cross-Section of New Bay of Car Shop Showing Details of Construction and Dimensions.

COST DATA.

The following cost data apply to the work done under the conditions already described:

Moving old equipment, tracks, blacksmith shop and oil house, and miscellaneous labor not hereinafter provided for		$ 726.36
Excavation outside of buildings and hauling away the dirt on electric cars, 910 cu. yds. (cost per cu. yd., about 60 cts.)		547.5
Excavation within the neat lines of the buildings, and hauling 1,000 ft. with teams to the dump, 3,086 cu. yds. (cost per cu. yd., about 61 cts.)		1,890.5
Shoring and patching old walls of old shop, equivalent to 30 M bricks (cost per M bricks, about $23.52)		705.4
New brickwork—		
Bricks, about 328 M, at $5.65	$1,852.00	
Lime, 245 bbls., at $0.785	192.50	
Sand, about 277 cu. yds., at $0.64	177.07	
Cement, about 283 bbls., at $1.60	452.69	
Total cost of materials for brickwork, 328 M (cost of materials per M, bricks, $8.15)		2,674.2
Total labor cost of laying bricks, 328 M (cost of labor per M bricks, $7.04)		2,309.0
(Cost of brickwork in wall, $15.19 per M.)		
Scaffolding, labor for all scaffolding (lumber included in forms)		255.0
Forms for concrete floors, girders and columns, 37 M ft. B. M. (cost per M ft. B. M., $19.10, which is equivalent to $0.64 per cu. yd. of concrete, 1,100 cu. yds.)		707.6
Nails, about 3,000 lbs., at 2.64 cts.; equivalent to about 11 cts. per cu. yd. of concrete (about 725 cu. yds.)		79.1
Labor on forms equivalent to $1.35 per cu. yd. of concrete (1,100 cu. yds.)		1,481.1
Total labor and material cost for forms, about 1,100 cu. yds., at $2.06 per cubic yard.		
Reinforcing steel—		
Kahn bars, about 29 tons, at $46.50	$1,346.00	
Hy-rib, about 360 squares, at $3.80	1,367.00	
Total cost of reinforcement		2,713.0
Cost of steel per cubic yard of concrete, $2.47.		
Labor on Kahn bars, about 29 tons, at $8.18	$237.73	
Labor on Hy-rib, about 360 squares, at $1.41	507.62	
Total labor cost, at 68 cts. per cubic yard (1,100 cu. yds)		745.

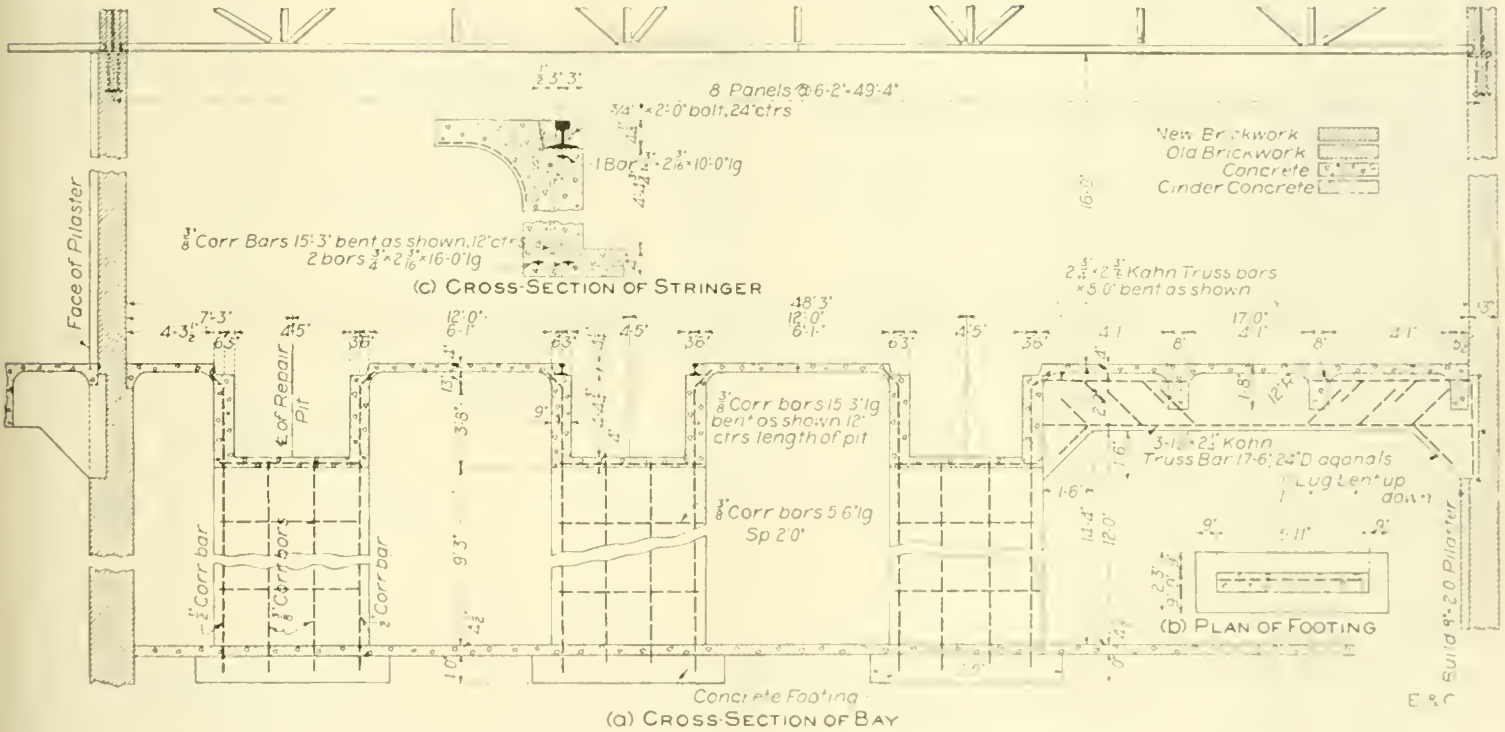
Total labor and material cost for reinforcement, about 1,100 cu. yds., at $3.15 per cubic yard.	
Concrete floors, pits, girders and columns—	
Crushed stone, about 602 tons, at $1.07	$ 644.14
Sand, about 367 tons, at $0.64	234.88
Cement, about 1,044 bbls, at $1.60	1,670.40
Cost of materials, about 765 cu. yds. concrete, at $3.33	2,549.42
Cost of labor, about 765 cu. yds. concrete, at $2.13	1,626.80
Total labor and material cost for concrete, about 765 cu. yds. of concrete, at $5.46 per cubic yard.	
Cement basement floor—	
Materials, 6,334 sq. ft., at about 6 cts.	385.26
Labor, 6,334 sq. ft., at about 5½cts.	347.12
Total labor and material cost, 6334 sq. ft. at about 11½ cts. per square foot.	
Concrete roof slab—	
Materials, 180 squares, 2½ ins. thick, at about $4.09	737.57
Labor, 180 squares, at about $4.03	726.16
Total labor and material cost, 180 squares, at about $8.12 per square.	
Waterproofing pitch for roof—	
11,600 lbs., at 1 ct. (6.5 lbs. per square foot)	
Material, about 180 squares, at 64 cts.	117.06
Labor, about 180 squares, at 36 cts.	64.96
Structural steel—roof trusses and purlins—	
Material, about 69 tons, at $50.70	3,499.00
Labor erecting, about 69 tons, at $7.61	525.39
Miscellaneous steel	68.15
Steel sash and glazing—	
12 curved head steel sash, 6 by 12 ft., at $30.83	370.00
384 lin. ft. Monitor sash, at $1.00	384.00
12 sash, 3x6 ft.	104.00
2 steel doors, 3x7 ft.	80.00
Wire glass, 2,750 sq. ft., at 14 cts.	385.09
"Puttyr" brads, clips, etc., at 1⅓ cts.	36.70
Total materials, 2,750 sq. ft. sash, at 48 cts. per square foot.	
Labor, 2,750 sq. ft. sash, at 4 cts.	110.00
Cost per square foot, in place, 52 cts.	
Sash operating device, 384 lin. ft., at 81 cts.	310.00
Labor erecting 384 lin. ft., at 9½ cts.	36.50
Rolling steel shutters, 7 shutters, material and erection (1,230 sq. ft., at 76.7 cts. per square foot)	944.00
Vitrified clay tile for drainage	109.10
Labor laying tile drains	90.16
Elevator (capacity 2½ tons), erected	1,200.00
Trolley troughs, 1,200 lin. ft.—	
About 5,500 ft. B. M. lumber	121.78
155 barn hangers and fittings	87.32
Labor, at $23.50 per M ft. B. M. (1 cts. per ft.)	129.93
Storeroom shelving—	
About 24 M ft. B. M. lumber, at $21.80	522.51
Nails, bolts and incidentals	21.02
Labor, 24 M ft. B. M., at about $27.00	651.27
Lowering sewer—	
Brick, sand, cement, crushed rock	77.50
Labor	597.52
Cost of lowering sewers and repairs, about 150 ft. (average depth, 4 ft.), at $4.50 per linear foot.	
Accidents and injuries	291.68
Tools and equipment	255.12
Total cost of materials	$18,542.26
Total cost of labor	13,857.94
Total cost of building	$32,400.20

DISCUSSION.

The rates of wages paid were: Laborers, $1.25 and $1.35 per day; carpenters, $2.50 to $3.00 per day, and brick masons, $4.00 per day.

To make the above cost data of greater value in their application to other structures the following additional information concerning various items is given:

By referring to the foregoing data it will be noted that the concrete for the beams, girders, columns and 4-in. floor slabs cost $5.46 per cubic yard, in place. Of this amount $2.13 was for labor, which includes the unloading of the material from the cars, rehauling on our own cars, mixing by hand, delivering to the forms in wheelbarrows, and depositing in the forms. This work was done in small sections, a large part of it being done at irregular intervals in a shop where car repairs were being made. Under these conditions the costs appear to be very reasonable.


8 Panels @ 6-2"-49'-4"
¾" × 2'-0" bolt, 24" ctrs
1 Bar ¾" · 2 3/16" × 10'-0" lg
⅜" Corr Bars 15'-3" bent as shown, 12" ctrs
2 bars ¾" × 2 3/16" × 16'-0" lg
New Brickwork
Old Brickwork
Concrete
Cinder Concrete
Face of Pilaster
℄ of Repair Pit
⅜" Corr bars 15 3" lg bent as shown 12" ctrs length of pit
⅜" Corr bars 5 6" lg Sp 2'0"
½" Corr bar
⅜" Corr bars
½" Corr bar
2 ¾" × 2 ¾" Kahn Truss bars × 5 0" bent as shown
3-1 ¼" × 2 ¼" Kahn Truss Bar 17-6", 24" Diagonals
Lug bent up down
Build 9" × 2 0 Pilaster
Concrete Footing
48'3"
12'0"
6'-1"
7'-3"
4'-3½"
17'0"
E & C

(c) CROSS-SECTION OF STRINGER

(b) PLAN OF FOOTING

(a) CROSS-SECTION OF BAY

Fig. 4. Cross-Section of Old Bay of Car Shop Showing Details of Construction and Dimensions.

The cement floor in the basement was placed at a labor cost of 5½ cts. per square foot. The floor was laid off in 3-ft. squares, and one or more square was placed at a time, one-half of the basement being used as a machine shop during the entire operation.

The 2½-in concrete roof slab was constructed under similar conditions, except that the concrete was hoisted to the roof by hand, in buckets, the labor cost being $4.03 per square.

The waterproofing for the roof consisted of a single application of roofing pitch, brushed on while hot. This is an unsatisfactory finish as: (a) the pitch creeps toward the gutters in hot weather; (b) a 2½-in. roof slab is certain to have expansion cracks, and in cold weather the pitch "checks," allowing water to penetrate it; and (c) the pitch flakes off during cold weather, and the water percolates through the thin slab. The company expects to apply a tar paper and gravel roofing in the near future.

The roof trusses and purlins were erected with a gin-pole by men not familiar with such work.

Of the 327 M. new bricks used fully 30 per cent were used to patch and top out the old walls, at a total labor cost of $7.04 per M. from freight car to wall.

The construction of the storeroom shelving shows a high labor cost ($27.00 per M. feet B. M.), but this cost is not unreasonable when it is considered that this was all bin work, fully 50 per cent of the lumber being cut approximately to 2-ft. lengths, and that all work was scribed to the rough brick walls.

The cost of lowering the sewer ($4.50 per linear foot) is hardly comparable with other work, as it was a special problem.

The volume of the two bays is approximately 492,000 cu. ft., which gives a cost for materials of about 3.8 cts. per cubic foot, and for labor of about 2.8 cts. per cubic foot—a total cost of 6.6 cts. per cubic foot.

There are approximately 27,000 sq. ft. of floor space in the two bays. Of this area, about one-fourth is storeroom space, about one-fourth is car storage space, and about one-half is used as a car shop. Expressing the costs in terms of floor area we have the following: For materials, $0.69 per square foot; for labor, $0.51 per square foot—a total cost of $1.20 per square foot.

The electrical work and the trolley work were handled by another department, and no costs are available.

The car barn, the pits, and the shops are served by a ladder track of 18 tracks, with 60-lb. rails. This work cost $7,500 for materials and $1,500 for its construction.

Electrolytic Iron.—Mr. Trygve D. Yensen of the Electrical Engineering Department of the University of Illinois, has for the past two years been making a special study of electrolytic irons with reference to their magnetic qualities. The aim has been to secure an iron of high permeability and small hysteresis loss and an iron more efficient than any now in use has been secured. A vacuum furnace was used for melting the iron, the pressure in the furnace being reduced to about one-half centimeter of mercury. Pure iron, when melted in the open air, will absorb a quantity of carbon and oxygen and so be of inferior quality, while iron melted in a vacuum does not absorb any of these elements, but remains pure. Various methods were used in cooling, tests being made to determine the effect of cooling slowly, plunging in brine, in liquid air, etc. Each iron was tested to determine the magnetic qualities, metallurgical composition, chemical composition, strength, and critical temperature. Much valuable information has been gained by these experiments, the results of which are soon to be published in the form of a bulletin by the Engineering Experiment Station of the University.

Results of Some Tests to Determine the Bearing Power of Piles.

In our issue of March 18, 1914, we published some data on the bearing power of soils and of piles, and also gave data on foundation work in Boston. To supplement these data the following tests on piling made at the new site of the Massachusetts Institute of Technology are of interest. The data were taken from a discussion of "Boston Foundations," by Mr. Charles T. Main, in the Journal of the Boston Society of Civil Engineers.

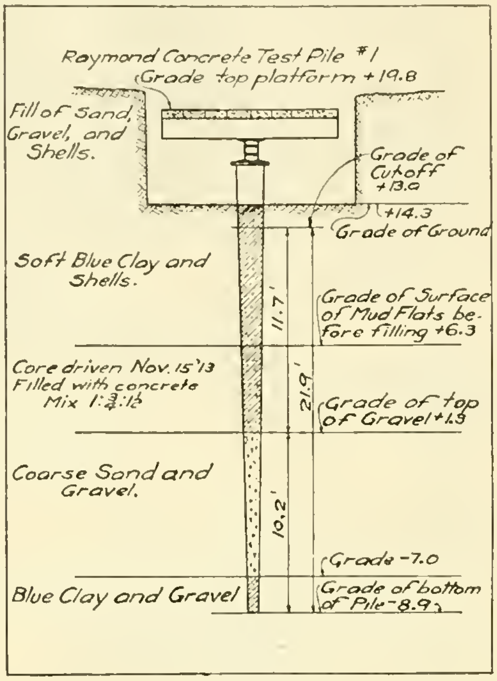
The results so far obtained at the new site of the Institute of Technology show that piles which have been properly driven, i. e., not broomed or broken, have a bearing power slightly in excess of that indicated by the commonly used formula:

$$P = \frac{2Wh}{p+1};$$

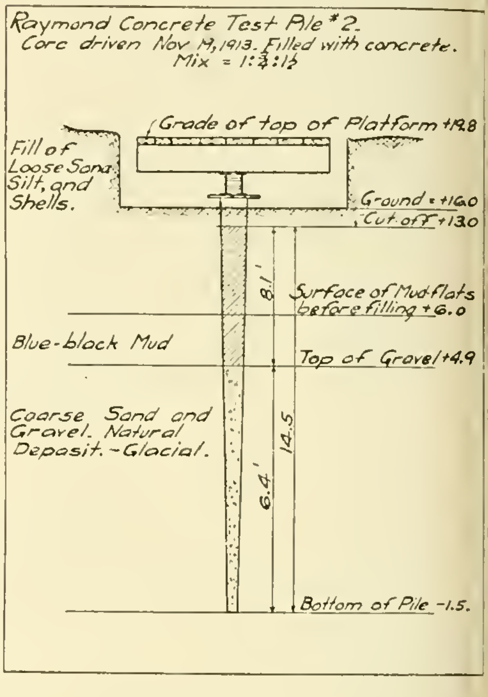
but those which have been brought up hard and have been crippled have shown a much less capacity under test loads than is indicated by the formula.

Tests and observation show plainly that spruce piles, without iron points, should be used cautiously in the Boston basin, due to the fact that the blue clay is often overlaid with a hard crust of gravel or sand. Reports of old piles which have been pulled in various parts of the city and have been found to be badly broomed and broken, as well as numerous tests made at the site of the Institute of Technology, show plainly that spruce is not suitable for anything except easy driving into loose sand or soft clay.

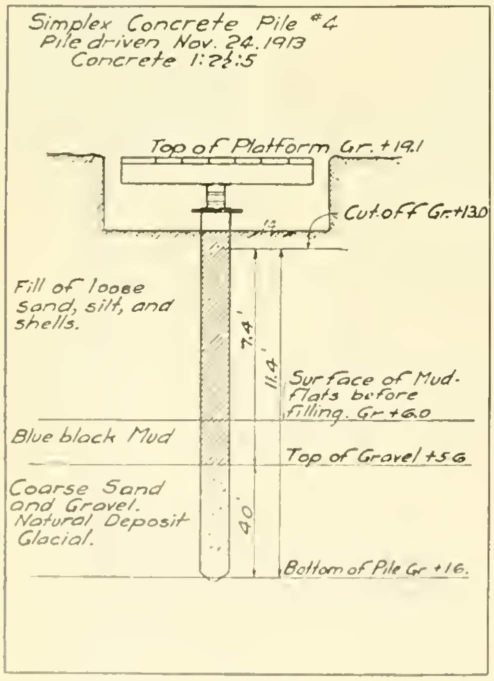
Piles broomed and broken near their points, but which are deeply embedded, may have a good bearing power from friction alone, but broken and broomed piles having little frictional support upon which to depend will settle badly and may fail under ordinary loads.


Raymond Concrete Test Pile #1
Grade top platform +19.8
Fill of Sand, Gravel, and Shells.
Grade of Cutoff +13.0
+14.3
Grade of Ground
Soft Blue Clay and Shells.
11.7'
Grade of Surface of Mud Flats before filling +6.3
Core driven Nov. 15 '13 Filled with concrete Mix 1:¾:1½
21.9'
Grade of top of Gravel +1.3
Coarse Sand and Gravel.
10.2'
Grade -7.0
Blue Clay and Gravel
Grade of bottom of Pile -8.9

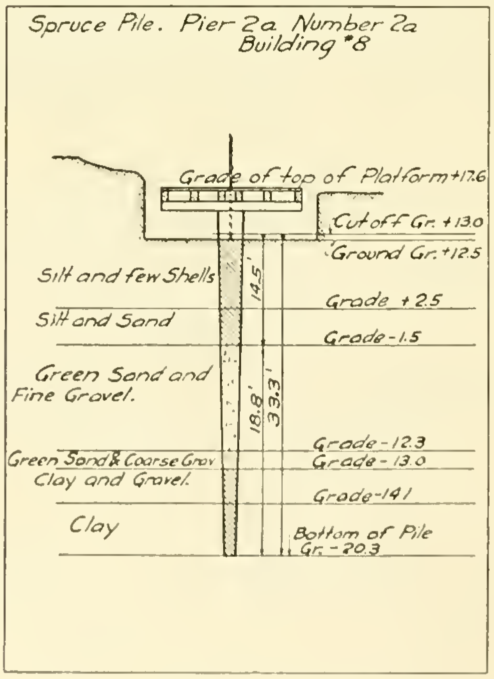
Pile No. 1.


Raymond Concrete Test Pile #2.
Core driven Nov. 14, 1913. Filled with concrete. Mix = 1:¾:1½
Grade of top of Platform +19.8
Fill of Loose Sand, Silt, and Shells.
Ground +16.0
Cut-off +13.0
8.1'
Surface of Mud-flats before filling +6.0
Blue-black Mud
Top of Gravel +4.9
Coarse Sand and Gravel. Natural Deposit.-Glacial.
14.5'
6.4'
Bottom of Pile -1.5.

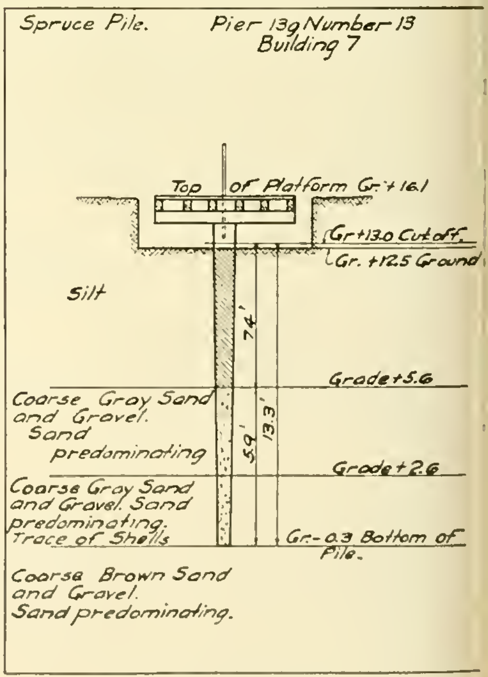
Pile No. 2.


Simplex Concrete Pile #4
Pile driven Nov. 24. 1913
Concrete 1:2½:5
Top of Platform Gr. +19.1
Cut-off Gr. +13.0
Fill of loose sand, silt, and shells.
7.4'
11.4'
Surface of Mud-flats before filling. Gr. +6.0
Blue black Mud
Top of Gravel +5.6
Coarse Sand and Gravel. Natural Deposit Glacial.
4.0'
Bottom of Pile Gr. +1.6.

Pile No. 4.



Pile No. 2, a.


Spruce Pile.
Pier 13g Number 13
Building 7
Top of Platform Gr. +16.1
Gr. +13.0 Cut off.
Gr. +12.5 Ground
Silt
7.4'
Grade +5.6
Coarse Gray Sand and Gravel. Sand predominating
13.3'
5.9'
Grade +2.6
Coarse Gray Sand and Gravel. Sand predominating. Trace of Shells
Gr. -0.3 Bottom of Pile.
Coarse Brown Sand and Gravel. Sand predominating.

Pile No. 13, g.

Fig. 1. Data on Tests of Five Piles at New Site of Massachusetts Institute of Technology.

The large number of piles tested show plainly that spruce piles fail to penetrate the hard crust, brooming and breaking badly, but that oak piles without iron points will penetrate the hard ground and will come practically to a standstill under the hammer without material damage. Any wood pile which must depend largely upon point bearing for its resistance to load should be oak or southern pine.

An interesting point is brought out by a study of the driving and testing records of some of the piles. These piles generally pass through a fairly hard fill of blue-black silty mud and shells before reaching the harder sand stratum in which they get most of their support. This fill gave considerable resistance to driving, and soon after the pile reached the sand it appeared (by the small penetrations under the hammer) that a satisfactory bearing power had been obtained. Such a conclusion, however, is unreliable, as the fill is subject to large future settlements due to decomposition, etc. The piles were therefore driven into the hard sand stratum to a depth which will give them a satisfactory support from that material alone.

The drawings and notes shown in Fig. 1 give the principal data for five of the piles tested. It will be noted that two of the test piles were "Raymond" concrete piles; one was a "Simplex" concrete pile; and two were spruce piles.

Table 1 (p 549) gives the essential data with reference to the five piles tested. The settlements for various test loads are given in this table

As noted before, most of the test piles showed that they could safely carry slightl higher loads than those given by the formula

$$P = \frac{2Wh}{p+1},$$

provided a small initial settlement is not detrimental. If a settlement of ¼ in. is assumed as a maximum the test piles have a factor of safety of from 2 to 3, the piles with the deep embedment as a rule giving the larger facto of safety.

Northwestern Road Congress. — Th Northwestern Road Congress will convene i Milwaukee, Wis., Oct. 28 to 31, 1914. An ex hibit of road building machinery, material, ap pliances, and accessories will be held durin the sessions. Delegates from Illinois, Iow Michigan, Minnesota, Wisconsin, North Da kota and South Dakota, representing state an county highway departments, boards of pul lic works, good roads associations, etc., wi participate in the congress.

TABLE I.—RESULTS OF TESTS OF PILES AT SITE OF NEW MASSACHUSETTS INSTITUTE OF TECHNOLOGY BUILDINGS.

Ref. No.	Type of pile.	Penetration in ground, ft.	Penetration in sand or gravel, ft.	Tip diameter, ins.	Weight of hammer, lbs	Fall of hammer, ft.	Penetration last blow, ins.	Test load, lbs.	Settlement, ins.
2a	Spruce	32.8	16.6	5½	2,150	10	2	8,990	.....
								13,760	3/64
								20,030	1/16
								28,210	5/64
								34,200	11/64
								40,040	11/64
								45,898	19/64
								51,000	19/64
								58,070	19/64
								62,360	19/64
13g	Spruce	12.8	5.8	6	2,300	10	2⅛	6,570	....
								11,860	1/32
								21,300	5/32
								26,480	5/32
								32,423	5/16
								37,560	⅜
								42,960	⅜
1	Raymond concrete.	23.2	10.2	8	No. 1 "Vulcan." Weight of moving parts, 5,000	2.25	5 blows per in.	41,050	.....
								46,070	1/16
								61,810	1/16
								81,840	5/64
								86,710	3/32
								90,890	⅛
								100,330	5/32
								125,310	3/16
				(Pile did not fail.)				144,770	¼
								171,750	11/32
								182,120	13/32
2	Raymond concrete.	17.5	6.4	8	5,000 No. 1 "Vulcan."	2.25	4 blows per in.	29,650	.....
								44,420	1/16
								59,360	⅛
								110,630	3/16
								122,280	¼
				(Pile did not fail.)				126,700	5/16
								131,610	⅜
								158,104	13/32
								169,980	35/64
								175,870	19/32
4	Simplex concrete..	15.6	4.0	16	3,300	12	¼	42,490	1/64
								61,280	¼
								85,800	3/16
								95,910	3/16
				(Pile did not fail.)				133,790	3/16
								153,810	¼
								177,020	9/32
								187,270	11/32
								192,270	7/16

Composition and Construction of Various Types of Shop Floors.

The essential requirement of shop and freight house floors is a good, hard wearing surface which is level, smooth, easy on the feet, easy to truck loads over, and capable of carrying heavy loads. The following data on different types of floor construction for shop floors were taken from the Bulletin of the American Railway Engineering Association, Vol. 15, No. 163:

CINDER OR GRAVEL FLOOR.

A type of floor in general use, and one that is best adapted for blacksmith shops and foundries, is shown in Fig. 1. It is made by filling in the space between foundation walls, preferably with sand or gravel, and bringing the filling up to the required grade, thoroughly compacting it as it is placed. The filling should be well flooded, rolled and tamped. A five to ten-ton roller should be used where possible. The minimum depth of the finished floor should be 8 ins., and if the top surface of the ground is soft it should be removed below this depth. For a top wearing surface hard screened cinders or stone screenings should be used to a depth of about [illegible] ins., and these should be thoroughly wet down and rolled to a firm, hard surface. Where clay is available it often can, with advantage, be mixed with the top surface and rolled into place. This makes a hard and more compact surface. Crude oil, when mixed with the top surface, also tends to harden it, and helps to prevent the wearing surface from becoming broken up.

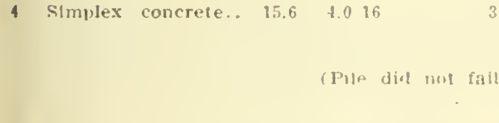


Fig. 1. Cinder or Gravel Floor, Especially Suitable for Foundry or Boiler Shop.

This type of floor is often used where an inexpensive floor is required, or where on account of a heavy fill inside of foundation walls a more expensive floor would fail due to settlement. It is not well adapted to trucking, and often an industrial track about 2 ft. wide with small push cars of a close-planked runway may be desirable where the most material has to be moved. Special foundations are necessary for all machinery.

PLANK FLOOR ON CINDERS OR GRAVEL.

A plank floor on cinders or gravel, shown in Fig. 2, is often found desirable where a heavy fill inside of foundation walls is required, where settlement may occur, and where the type of floor shown in Fig. 1 would not be satisfactory on account of the volume of trucking required or on account of the necessity of gathering up and saving scrap material, as in a machine shop. It consists of planking, spiked to sleepers resting on the filled material between the foundation walls. The filling, preferably sand or gravel, should be settled as mentioned in connection with Fig. 1, and should be brought up to within 9 ins. of the finished floor grade. On this should be placed 6 ins. of cinders, gravel, or other material of a porous nature, in the top surface of which 3x6-in. sleepers are embedded, spaced about 3 ft. on centers. They should be laid with [illegible] broken joints.

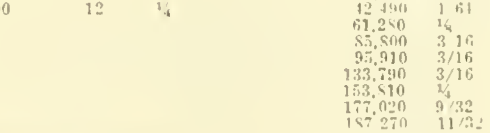


Fig. 2. Plank Floor on Cinders or Gravel.

This type makes a fairly good working surface which will last at least four years, at which time all settlement should have taken place in the filling and a better type of floor can be used. Long-leaf yellow pine will last longer than short-leaf yellow pine, but it will cost more. Fir and hemlock will longer resist decay than will short-leaf yellow pine, but they will not wear as long, and are not as good as long-leaf yellow pine. Special care should be taken to have the sleepers and plank thoroughly seasoned. For this reason it often is advantageous to get the lumber early on the site of the work, stack it and allow it to season. Additional life may be obtained, if desired, by creosoting the sleepers, or by creosoting both the sleepers and the plank. A cinder bed under the sleepers will give a little longer life than sand or gravel.

Special foundations are necessary for all machinery, and where jacking is done.

WOOD BLOCK FLOORS.

Wood block floors (see Fig. 3) are often used, and have the following advantages. They can be easily repaired; they are easy to work and truck on; and they do not damage falling tools. They need a concrete base to distribute any heavy loads which may bear on a few blocks only.

The filling between the foundation walls is done as is mentioned in connection with Fig. 1, and it includes a 6-in. bed of compacted cinders. On the cinders is laid a 6-in. course of 1:3:5 concrete. Steel reinforcement may often be placed in the concrete to advantage, particularly over soft spots in the filling or where heavy loads are apt to be placed. The reinforcement should be placed either near the top or bottom surface of the concrete, depending upon local conditions. Sand should be spread over the concrete, and should be brought, with a board or a template, to a uniform thickness which, when compacted, will amount to 1 in. On the sand bed there should be placed the wood blocks, which should be of an even thickness of at least 4 ins. The blocks should be cut across the grain so that they can be laid with the ends of the fibre exposed to wear. They should be uniform in width, but may have a variable length, although blocks of a uniform length can be laid quicker and more cheaply.

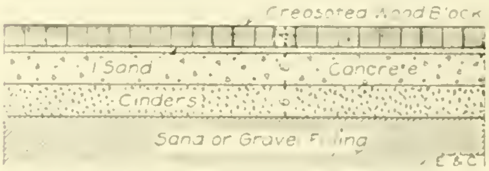


Fig. 3. Wood Block Floor on Concrete Base.

Wood blocks should be creosoted, and may be made from any material suitable for such treatment. Generally, however, the blocks are of short-leaf yellow pine, although long-leaf yellow pine blocks give the greatest wear.

The blocks should be laid with the fiber vertical and with close joints, with at least a 2-in. lap. Expansion strips 1 in. in thickness should be placed every 50 ft. across the building and at the sides of the building, or at any break in the floor surface. The blocks should be tamped or rolled to an even surface; the joints should be filled to within 1 in. of top surface with sand, and the remainder of joints should be filled with No. 2 [illegible] pitch. Immediately after placing the pitch, hot dry sand or gravel should be spread over the blocks to take up the surplus pitch.

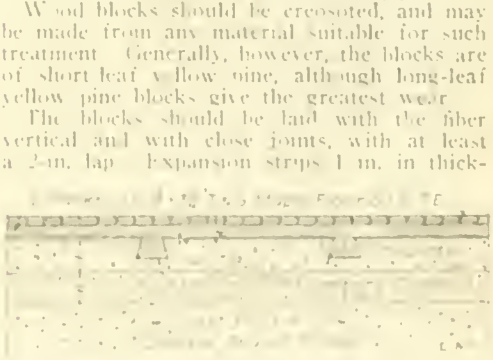


Fig. 4. Plank Floor on Concrete Base.

Creosoted planks are sometimes used in place of the concrete base. The resulting floor is not nearly as good, however, as one with a concrete base, and it is generally equally costly.

Special concrete foundations are necessary for heavy machinery, but for small machinery the foundations may be built up from the concrete base of such a size as may be required for the setting of the machine.

Scrap lumber (oak or yellow pine) is often used for shop floor blocks. The mill can take cuttings and condemned lumber, and saw it up at odd times. Consequently, such blocks cost practically nothing for material and very little for labor. Such floors are often laid directly on the filling or on planks, but they do not last over four or five years, and care must be taken to provide more expansion joints than with creosoted blocks.

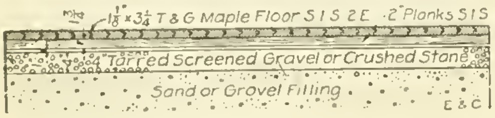

1⅛"×3¼ T & G Maple Floor S 1 S 2 E
2" Planks S 1 S
4" Tarred Screened Gravel or Crushed Stone
Sand or Gravel Filling
E & C

Fig. 5. **Wood Floor on Tarred Screered Gravel or Crushed Stone.**

Hexagonal blocks are sometimes used. They should be not less than 5 ins. nor more than 7 ins. deep, and all blocks for one job should be of the same size. Blocks of this type have no particular advantage, except that they are more stable, as they have more friction on the side surfaced and are not so easily tipped up at the corners.

A wood block floor with a concrete base is generally fully as expensive as any good type of floor, and often has to be relaid due to buckling.

ASPHALT BLOCK FLOORS.

Asphalt blocks, about 4x12x4 ins., are sometimes used to advantage, as they come on the job all ready for laying, and can be laid like wood blocks or like brick. They do not need expansion joints, nor does the laying of them require any skilled supervision. They do not heave, they stay smooth, and they wear slowly without chipping, except that where there is continuous dripping of oil, as directly under a vise, they soften and wear faster than at other points. They can be more easily taken up and repaired than other types.

PLANK FLOORS ON CONCRETE.

Figure 4 shows a wooden floor with a concrete base. It is a good type of floor, as it gives a fine surface on which to work or over which to truck. However, it is expensive.

The filling and concrete base should be placed as for a wood block floor, as mentioned in connection with Fig. 3, except that in the top surface of the concrete there should be embedded 4x6-in. creosoted sleepers. On these sleepers should be laid 2-in. planks, dressed to an even thickness and width. They should be laid with running broken joints. On the planks should be laid a top wearing surface of 1⅛x3¼-in. D. & M. maple flooring with ends matched, laid parallel to the direction of the maximum trucking, and with running broken joints. The flooring should be end-matched and bored for nailing, for which there is little, if any, extra cost. A square-edged floor may be used. It costs less, but is not quite so smooth, and will require more attention to maintain a good surface. It is especially desirable that the 2-in. planks should be thoroughly seasoned, and for this reason they should be brought on the site of the work early, stacked and allowed to season.

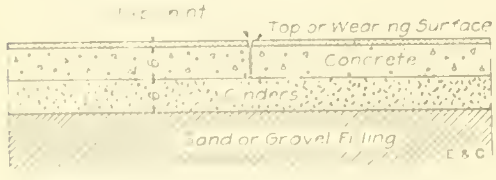


Fig. 6. **Concrete Wearing Surface on Concrete Base.**

This type of floor should ordinarily last from 10 to 12 years, and generally fails from dry rot of the sleepers and the underfloor. Additional life may be obtained by creosoting the sleepers and the underfloor, and by giving the top surface of the finished floor a good mopping of hot linseed oil, which also tends to lessen buckling.

Light machinery may be lag-screwed directly to this floor, and only heavy machinery need be provided with special concrete foundations extending lower than the concrete subfloor.

WOOD FLOOR SET IN PITCH AND TAR COMPOSITION.

Figure 5 shows a wooden floor with a concrete base, the wooden sub-floor being set in a top coat of tar and sand, spread over the concrete. Either Portland cement concrete or a tar concrete can be used as a foundation. This is a more permanent type of floor than that shown in Fig. 4, unless in the latter case both the sleepers and the underplank are creosoted.

Where Portland cement concrete is used it should be laid as shown in Fig. 4. Where the tar concrete is used on the compacted filling between foundation walls there is laid 4 ins. of coal-tar concrete. The foundation for this concrete should consist of either 4 ins. or 6 ins. of screened gravel or crushed stone, none of which should exceed 2½ ins. in the longest dimensions or be less than ¼ in. in size. It should be mixed with special sub floor tar (minimum amount stated below), so that it will compact under a roller after being spread evenly in place. It should then be rolled until the stones do not creep under the roller. The tar for this course may be heated to not more than 200° F., and in cold weather the stone should be slightly heated, so that the tar will mix with the stone and the stone will spread evenly. The roller used for this work should weigh not less than 300 lbs. per foot of length. The amount of tar used in the foundation should not be less than 6 gals. per cubic yard of 2½-in. to 1-in crushed stone; 9 gals. per cubic yard of 2½-in. to ¼-in. crushed stone; 7 gals. per cubic yard of coarse screened gravel; or 10 gals. per cubic yard of fine screened gravel.

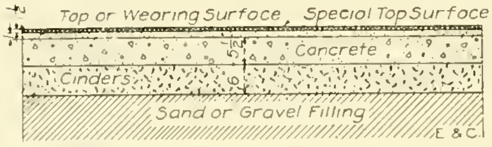


Fig. 7. **Special Top Surface on Concrete Base.**

If the mixing is done with a machine 25 per cent less tar will be required.

The top coat over the concrete should consist of a fine sand thoroughly mixed with specially prepared tar in the proportion of not less than 50 nor more than 60 gals. of tar to each cubic yard of sand. The sand should be thoroughly dry before mixing, and neither tar nor sand should be hotter than 225° F. when being mixed together. If the materials are hot enough so that a thick white smoke arises from the mixture, 5 gals. more of tar for each cubic yard of sand should be used. This mixture should be spread evenly over the foundation to a thickness of 1¼ to 1½ in. (so that it will compact to 1 in.) It should then be leveled with a straight edge and should be followed closely with the plank.

The top coat mixture may be tested as follows: If 10 to 20 cu. ins. of mixture, at a temperature of 175° to 200° F., be placed in a tight vessel to a depth of not less than 1 in. and "patted" it should be deemed that the mixture contains sufficient tar if tar shows on the surface.

Planks having a thickness of 2 ins. should be laid on the soft material, and should be bedded on it by hammering until the proper stability is obtained and the planks brought to a proper level and toe-nailed together. If, after hammering, any plank is below the proper level it should be taken up and more of the top coat applied.

In order to insure the use of seasoned planks, it is desirable that the planks should be on the premises as long as practicable before being laid and stacked, so that they will have the best opportunity to season, and should be covered with boards to protect them from rain. If green planks are used and are covered with a hardwood floor dry rot may result.

Cinders make a foundation in every particular as good as stone, but they require at least 15 gals. of this special tar per cubic yard and need far more rolling to properly compact them.

Sand may also be used for the foundation, but at least 20 gals. of tar will be required per cubic yard, and a different special tar must be used. The sand will also need to be heated before the tar will mix with it properly. If desired, either a cinder or a sand foundation may be compacted with a rammer instead of with a roller.

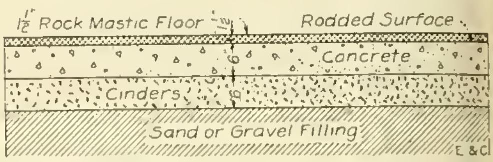


Fig. 8. **Rock Mastic Surface on Concrete Base.**

Light machinery may be attached to this floor without any additional foundations, but for heavy machinery special concrete foundations must be provided.

CONCRETE FLOORS.

Figure 6 shows a cheap and fairly permanent floor. It is easy to truck over, is easily cleaned, is sanitary, and has the advantage that no special foundations have to be provided, except for the heavier types of machinery. Light machinery is simply bolted to the floor. Industrial tracks may be easily and cheaply installed in the floor, with the head of the rail flush with the top surface. This floor, however, easily damages falling tools, it is hard to work on it, and it quite easily becomes worn in spots.

In building it, make a fill as for the other types of floors, and over the filling spread about 6 ins. of hard screened cinders, properly compacted. Then lay a concrete floor 7 ins. thick of the same concrete proportions as given in connection with Figs. 3, 4 and 5, with the exception that the top or finished surface should be composed of 1 part Portland cement and 1 to 2 parts torpedo sand, troweled smooth to a sidewalk finish before the base has taken its initial set. Provision must be made for expansion by putting in alternate slabs with small V-shaped joints, these slabs having dimensions not over 8x16 ft.

Sometimes granite screenings are used instead of torpedo sand to give additional wearing life, although the cost is somewhat increased by their use.

Ordinary concrete floors are porous, and the constant wear results in granulation and abrasion. Holes are started which rapidly increase in size and gradually make the floors useless. The heavy loads, the trucking, and the constant hard usage make them wear unevenly and cause them to break up.

CONCRETE FLOOR WITH SPECIAL FINISH.

Figure 7 shows a concrete floor with a special top finish. It is designed to be more lasting than the usual concrete floor, as the special top is designed to stand harder wear and to keep the floor surface from becoming rough.

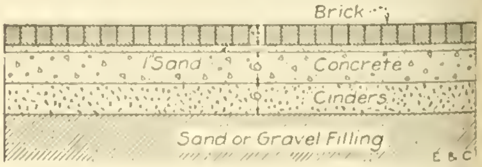


Fig. 9. **Paving Brick Floor on Concrete Base.**

There are several special materials on the market which are used for the top finish and which give good results. These special surfaces generally consist of some mineral powder mixed with other substances. When applied the particles of this powder expand, thus filling the porous places in the concrete and giving a dustproof, wear-resisting and waterproof floor of flint-like hardness. One advantage is that this topping can be applied after the base is set without materially damaging its efficiency. These special top dressings may also be used to advantage in patching old damaged concrete floors.

ASPHALT FLOORS.

Figure 8 is considered to be an ideal floor for shops if properly laid with the correct materials and mixtures. Experienced supervision must be employed to get the best results. Similar floors are still in service and in fair condition after having been laid 25 years.

Floors of this type will greatly outwear other types of floors. They give the qualities which are desirable in a floor, and are without the objectionable features which have been mentioned in connection with other floors. They are easy to walk on and to truck over, and the greater the traffic the more dense and durable they become. They do not grind away materially under truck traffic, do not easily wear uneven, do not easily crack or disintegrate, are noiseless and dustless, and can be kept clean with a broom or mop, or by an occasional flushing with a hose. They are sanitary, are waterproof and fireproof, and are easily repaired. The filling and concrete sub-floors are laid in the same manner as are other types of floors. The top of the concrete should be drawn out under a straight edge, and should be struck off but not troweled.

Mastic blocks should be delivered on the ground, plainly marked with the name of the brand, and should be broken up before being placed in the mastic boiler. Asphalt flux should then be added, and both materials should be allowed to cook until the mastic blocks are entirely melted. Washed torpedo gravel, torpedo sand, crushed limestone, or granite, in the proper percentage to give the required hardness, should then be added and thoroughly mixed into the mass by iron stirring rods, and the temperature of the mixture brought to 450° F. The material must be constantly stirred to prevent burning, and then should be removed from the kettles in all-iron wheelbarrows or oak buckets and taken to the work as required.

The gravel or stone must be thoroughly dry before being put into the mastic. It should be a clean, well-graded material which contains no particles larger than would pass through a ¼-in. mesh.

Native bitumens do not give as good results as do the imported mastics.

BRICK FLOOR.

Figure 9 shows a brick floor with a concrete base. Such floors are easily repaired, are easily cleaned, are sanitary, and are fairly cheap, but they are hard to truck over, hard for men to work on, and are hard on falling tools. The filling, the concrete base, and the 1-in. sand cushion are placed as for a wood-block floor. The brick floor is then laid on this base. The bricks should be vitrified, repressed pavers, and should be laid edgeways and carefully tamped or rolled to insure an even top surface. The intervening spaces between the bricks should be filled with Portland cement and sand of a 1:1 mixture, which should be poured as a thin grout. It should be followed up with a stiffer mixture and then covered over with sand. Expansion joints are necessary, as for creosoted blocks, but the joints need not be so large.

Special foundations for machinery must be provided, as with a creosoted block floor.

Roads and Streets

Organization for and Methods and Costs of Constructing Concrete Roads in Milwaukee County, Wis., in 1913.

Contributed by A. J. Kuelling, County Highway Commissioner, Milwaukee County, Wis.

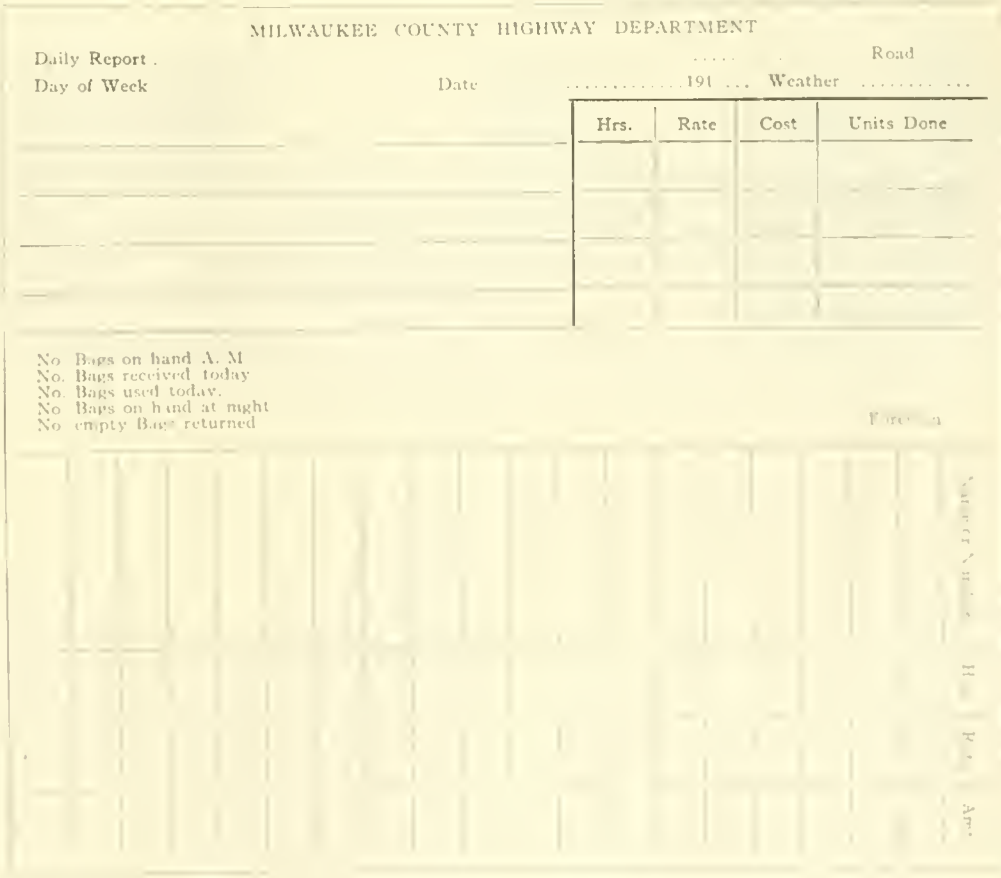
Milwaukee County, in which the city of Milwaukee is located, is in the glaciated section of southern Wisconsin. The topography varies from comparatively level near the lake to rolling a few miles inland. In the west end of the county some grades are as great as 10 per cent, although the steepest grade upon which concrete has been placed is 6 per cent. The soil varies from a very light, sandy loam along the lake shore, to a heavy clay a few miles toward the west.

It is therefore seen that the concrete roads described have been built under a wide variety of conditions. The old roads were so rough in profile and in such poor condition as regards the surface that the roadbeds were in all cases entirely destroyed. The writer believes that this destruction of the old hard surface and the consequent uniform texture of subgrade obtained has tended to minimize the number of cracks in the pavements.

It was only after careful consideration that a concrete pavement was adopted by Milwaukee County for use on these rural roads. Not only was the present heavy, mixed traffic (common near a large city) taken into consideration but the certainty of an immense future motor-driven traffic (especially for freight) was also given careful consideration. Naturally water-bound macadam and gravel were not considered, while bituminous macadam or brick on anything but a concrete base were not considered suitable and the latter types on a concrete base made the cost objectionable. It was understood that there are both advantages and disadvantages in the concrete road, but after considering these and recognizing the fact that the concrete road, no matter what its length of life, would always be available as a foundation for any kind of surfacing, this type was adopted.

All of the work is done under the Wisconsin state aid highway law, whereby all contracts and all work must meet the approval of the State Highway Commission. Under this law the town and county, or the county alone, pays two-thirds of the cost of construction while the state provides the remaining one-third.

Extent of Work.—The amount of work accomplished in this county during the past year is probably as large, if not larger, than that done in any other similar community in the middle west. There were constructed 22 miles of concrete road, all of which were either 16 ft. or 18 ft. in width with the exception of about one-half mile, which was 24 ft in width. Some idea of the magnitude of this undertaking can be gained from the fact that about 2,500 carloads of material were used and over 300,000 bags of cement.



MILWAUKEE COUNTY HIGHWAY DEPARTMENT

Daily Report . ..... Road
Day of Week Date ....191 ... Weather ....

Hrs.	Rate	Cost	Units Done

No Bags on hand A. M
No. Bags received today
No. Bags used today.
No Bags on hand at night
No empty Bags returned

Foreman

Fig. 1. Daily Report Form to be Used in 1914. (5 in. x 8 in. in Size.)

Organization.—The organization for handling this work was as follows:

The highway commissioner, who had immediate supervision of all of the work; the assistant highway commissioner, who attended to the office work; the chief clerk, who attended to the correspondence, payrolls, writing of checks and distribution; the traffic man, who attended to the spotting of cars, keeping a supply of materials on the jobs and the computation of all freight; the superintendent of construction, who supervised all of the jobs, and a surveying party of three members who made the surveys, did all of the platting and figured all of the yardage. A master mechanic was also employed to look after the $10,000 worth of machinery owned by the county.

Method of Conducting Work.—Part of the work was done by contract and part by force account under the direct orders of the highway commissioner. The foreman of each force account crew and the inspector on each contract job turned in a daily report on a self-addressed postal card. In Fig. 1 is shown a type of daily report, which will be used during the coming season.

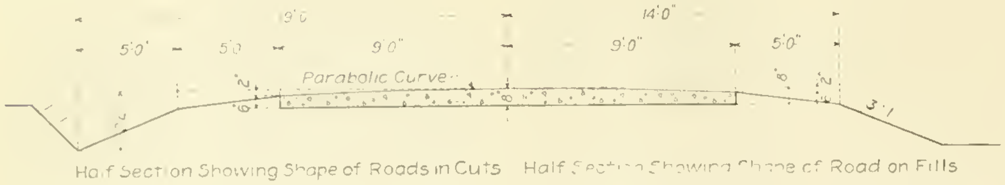

9'0"
14'0"
5'0"
5'0"
9'0"
9'0"
5'0"
Parabolic Curve
3:1
Half Section Showing Shape of Roads in Cuts
Half Section Showing Shape of Road on Fills

Fig. 2. Standard Cross-Section for 18-ft. Concrete Roads Built in Milwaukee County in 1913.

No special difference has been noted in the cost of construction between the two methods, but it is believed that a slightly better quality of work was done by force account crews.

Preparation of Roadbed.—After the roads had been surveyed and the plans drawn up, the final grade line was so placed that the cuts and fills balanced. A planimeter was used in computing the area of cross-section at each station.

TABLE II. RATE OF COMPENSATION OF LABOR—10-HOUR DAY.

Item.	Cost.
Foremen, per month	$125.00
Engineer, hauling and mixer, per day	3.00
Concrete finisher, per day	3.00
Ordinary labor, per day	2.50
Teams	6.00

The material encountered varied from sand to stiff clay. Several of the roads were old plank toll roads and considerable difficult grading was encountered as a result of the old planks remaining in the road. Where possible, tractor engines were used for drawing plows or grader. The use of these engines furnished all the rolling that the roadbed received, as it is not considered practical to keep a supply of material ahead and at the same time roll the subgrade immediately before placing of the concrete. It is also believed by the writer that the subgrade can be rolled too much. Practically all of the grading was done either with wheelers or slip scrapers. It is believed advisable to grade for a considerable distance ahead in order to give the ground sufficient time to thoroughly settle before the concrete is placed. Where underground springs were encountered, tile drains were placed t care for the underground water.

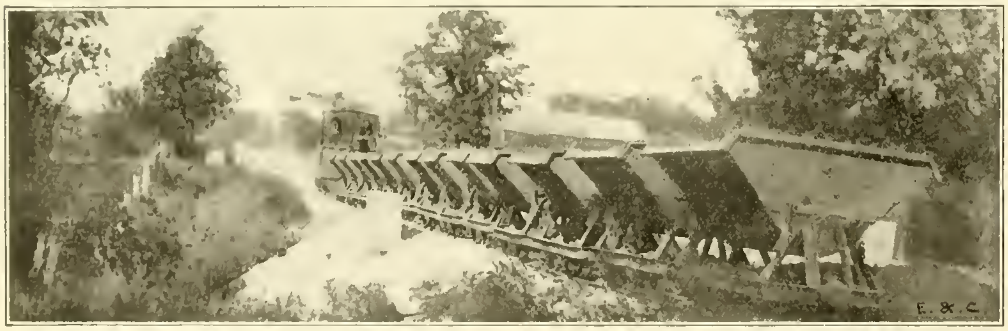


Fig. 3. Industrial Railway for Hauling Material on Loomis Road.

TABLE III. ORGANIZATION OF CONCRETE MIXER CREW, MILWAUKEE CO., WIS.

Men.	Duty.	Cost.
1	Foreman	$5.00
1	Assistant foreman	3.50
1	Mixer engineer	3.50
1	Fireman	2.75
1	Finisher	3.00
3	Wheeling concrete @ $2.67 per day	8.00
2	Handling cement @ $2.50 per day	5.00
3	Handling sand @ $2.50 per day	7.50
8	Handling stone @ $2.50 per day	20.00
2	Forms and joints @ $2.50 per day	5.00
1	Evening and tamping	2.50
1	Covering and sprinkling	2.50
1	Pump man	2.75
1	Water boy (cared for cement sacks)	1.50
1	Watchman	2.25
28 men.	Total labor cost per day	$74.75

Average, 500-600 sq. yds. (95-115 cu. yds.) per day.

The cost of grading is given in Table I. Their cost includes the hand trimming of all slopes and the trimming of ditches. Table II shows the average rate of compensation of labor during the season.

TABLE I.—COST PER CUBIC YARD OF EARTHWORK ON CONCRETE ROADS BUILT IN 1913.

Item.	Fond du Lac.	Port Washington.	Blue Mound.	Loomis.	Chicago No. 4	Chicago No. 3.	Janesville Nos. 2 and 3.	Green Bay.	Mukwanago.
Labor	$0.69	$0.68	$0.65 (County) 0.67 (contract)	$0.71	$0.68	$0.67	$0.77	$0.84	$1.05
Materials	0.06	0.01	0.01	0.01	0.02	0.01	0.04	Contract	Contract
*Workmen's losses	0.02	0.02	0.03	0.02	0.02	0.02	0.02	Contract	Contract
*Machinery repair and depreciation	0.02	0.02	0.03	0.02	0.02	0.02	0.02	Contract	Contract
Cost per cu. yd	$0.78	$0.73	$0.72 (County)	$0.75	$0.73	$0.71	$0.81	$0.84	$1.05
Number of cu. yds. of excavation	6,524	4,000	5,736 (County) 9,402 (contract)	6,500	4,500	1,998	15,067	1,527	1,002

*Prorated from totals for the year.
Average cost per cubic yard for the whole season......$0.76
Total number of cubic yards for the whole season......56.256

Cross Section.—The roads were graded to the cross section noted in Fig. 2. It will be noted that the subgrade is flat and that the crown of the road is cared for by making the concrete thicker in the center of the pavement. The material for the earth shoulders was thrown up in a windrow along the berm of the road so as to be easily available after the concrete was placed.

Fig. 4. Method of Setting Forms and Supplying Water to the Concrete Mixer.

All work done was of the one course type, which was 6 ins. thick at the edge and 8 ins. thick in the middle, with the exception of one road which was 5 ins. thick at the edge and 7 ins. in the middle. On most of the roads gravel shoulders were constructed, but for the work the coming season they will be abandoned except in special cases.

Materials.—The concrete mixture used during 1913 was a 1:2:3½, although this was varied a little where the voids in the aggregate were found to be very low. All cement used was tested by contract at the mills, an occasional test being made in the laboratory of the highway commissioner. Practically all sand and stone were washed, and in every case the sand and stone were brought onto the job separately. A large number of tests were run on both sand and stone to determine: (1) the per cent of voids, (2) the per cent of clay or silt, (3) the per cent of silicious material in the sand, and (4) the nature of the stones composing the larger aggregate. These tests were made before the work was commenced and from time to time upon samples obtained from the job. The sand was graded from ¼ in. down to not more than 3 per cent passing a 100 mesh screen. The larger aggregate was in most cases graded from 1½ ins. to ¼ in. It is believed that unless the aggregate is thoroughly tested, poor results are liable to be obtained.

In all cases the cement was furnished to the contractors by the county, contracts being entered into with local dealers for furnishing and delivering cement on the jobs. Careful attention was given to the care of empty cement sacks. In all cases these were tied into bundles of 50, each with a tag attached showing the name of the dealer furnishing the cement, the name and number of the road and the name of the contractor. Lost cement sacks amounted to only 1½ per cent of the total number used.

Transportation of Materials.—One of the greatest problems in country road work is the transportation of materials to the job. A continuous rain will often delay the work for 24 hours or even longer, whereas in the city hauling may be commenced immediately after the rain. The method of hauling was quite variable. On some jobs teams alone were used, on others teams and steam tractors were used in conjunction, while on one job an industrial track was rented. The cheapest hauling was done on the Fond du Lac road with a combination of teams and steam tractor. The average haul on this road was almost two miles. One advantage that might be noted for tractor hauling is that the wagons leave the material in deep piles and therefore cause less loss due to material getting into the subgrade. Where tractors were used, the trains were pushed into place before being dumped, instead of backing the empty train out.

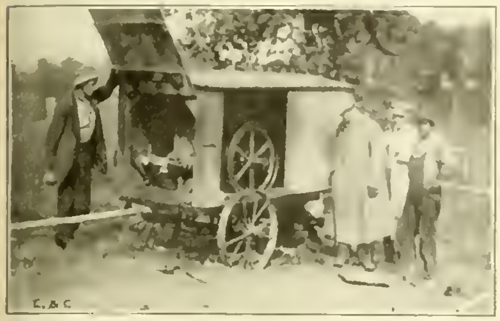


Fig. 5. Gasoline Pumping Outfit for Supplying Water for Construction.

It is believed that for hauling purposes tractors should have a beveled wheel in order not to disturb the crowned macadam roads over which much hauling must be done. Such beveling would also be advantageous to the engine as cases have come under the writer's observation where every spoke on the drive wheel was broken, due to the entire load being carried on the inner edge of the rim.

Material was placed on the road in two different ways: (1) the rows of sand and stone were placed close together and the teams delivering the cement were compelled to drive on the berm, (2) the sand and stone were separated and the cement teams were permitted to drive up the center of the road. It is highly advantageous to get a large quantity of material ahead in order to avoid any possible delays in shipping and also any delay in hauling. The longer the haul the more advantage the tractors will have over the teams.

Water.—Another problem that is often difficult in concrete road construction is the supplying of sufficient water, which averages about 4,000 gals. per day per crew. In all cases a 2-in. pipe line was laid from the nearest source of supply and where necessary, a small gasoline engine was supplied for forcing the water through the pipe. Unless the distance exceeds three miles, and the elevation exceeds 100 ft. a 2-HP. gas engine is sufficient to furnish the necessary power as there is very little friction in a 2-in. pipe supplying 4,000 gals. per day. It is advisable to have the engine connected with the pump by a belt in order to avoid any danger of breaking the cross-head on the pump. The most satisfactory safety device found for caring for excess pressure was the erection of a standpipe beyond the mixer. This standpipe was nothing more than one length of pipe turned into a tee.

Galvanized pipe is to be preferred, although it costs about 40 per cent more than black pipe. It can be used many more times and its salvage value is much greater. In cases where the pipe was injured in water freezing in it, an oxy-acetylene torch was used for welding with satisfactory results.

While erecting the pipe line, unions are placed about every 1,000 ft., tees with connections for 2-in. hose every 400 ft., and a valve about as often as the unions. Water in some cases was obtained from hydrants, while in others it was obtained from creeks or ponds. However, no contaminated water was permitted. For bringing water from the pipe line to the mixer, each crew was furnished with 225 ft. of 2-in. hose while for sprinkling purposes each crew was furnished with the same amount of 1-in. hose.

Mixing.—An efficient crew for use on country road work and about the average crew used on the work during 1913 is given in Table III.

Under favorable conditions, such a crew should mix and place about 600 sq. yds. of pavement. Due to the fact that the roads were comparatively narrow, wheel-barrows of large capacity were used in order to make the number of men ahead of the mixer as small as possible. A batch consisted of two bags of cement, 4 cu. ft. of sand on one wheel barrow, 7 cu. ft. of stone on two wheel-barrows. Such a combination makes fewer wheel barrows to dump, thus loading the hopper quicker and also lessening the chance for errors in getting the proper mixture for any batch.

Placing.—Forms were placed as far as possible in advance (usually about 300 ft.) in order to avoid any sudden change in the grade, as the grades are given only at intervals of 100 ft. The forms used were 2x6-in. planks with an angle iron facing at one edge. These were staked with iron pins and held firmly to line and to grade. The concrete was brought to place in some cases by spouts and in some by booms. Unless a two-course pavement is to be built the writer believes that the spout machine is preferable. In no case were horse carts used as the width is too narrow for convenient handling and the carts also prohibit the placing of concrete for a longer time after a rain than do either of the other methods of placing. Concrete was placed with as little handling as possible and was of such a consistency that there was no separation of mortar and stone. There was no tendency on the part of a contractor or foreman to mix too dry, while there was a tendency to mix the concrete too wet. As soon as the concrete was in place, it was struck off with an iron-shod board cut to the crown of the road. As small an amount of finishing as possible was done and the so-called finisher was instructed to merely remove any foreign matter, which in all cases will come to the surface, and to smooth the marks of the strike-off board, the idea being to leave the aggregate as near the surface as possible and not work any appreciable amount of mortar to the surface.

Curing.—When the temperature was over 90°, or when a hot wind was blowing, the concrete was protected with canvas until it could be sprinkled. As soon as it was sufficiently hard, the road was covered with earth from the sides and kept sprinkled for nearly a week. In some cases lawn sprinklers were used which were set from time to time by the water boy, thus avoiding the necessity of a man to do this work. Traffic was in all cases kept off at least three weeks, or a longer time than this, depending upon whether or not the weather was favorable for the hardening of the concrete.

Shoulders.—Where earth shoulders were built, the work was done by the mixer crew at times when material was lacking or machinery broken. Where gravel shoulders were built they, of course, were not constructed until after the road was thrown open to traffic.

Joints.—In all cases during 1913, armor plates were used with an asphaltic felt ¼ in. thick and the depth of the pavement. In Milwaukee County the spacing of the joints has been increased from 25 ft to 35 ft and in 1914 will be increased to 50 ft. The reason for this increase is the lack of cracks in the pavements already constructed. As before stated, the writer believes one reason for this lack of cracks is the thorough destruction of the old road, thus giving a uni-

Fig. 6. Method of Conducting Work—Laying, Finishing and Covering.

TABLE IV. COST OF CONCRETE ROADS BUILT BY FORCE ACCOUNT (1913).

	Name of Road								
	Fond du Lac.		Port Washington		Blue Mound.		Loomis.		
Item.	Cost per sq. yd. paved.	Total cost.	Cost per sq. yd. paved.	Total cost. Gravel	Cost per sq. yd. paved.	Total cost	Cost per sq. yd. paved.	Total cost.	Remarks.
Shoulders	$0.088	$1,847.43	$0.035	$ 369.82	$0.074	$3,391.17	$0.027	$ 751.29	Includes all labor and material.
Hauling and labor loading wagons	.326	6,830.88	.271	2,843.95	.428	19,551.28	.601	16,945.30	
Coal, oil, etc., for mixer and pump	.026	547.47	....	....	.018	819.31	.035	980.06	
Lumber, etc., for surfacing	.013	283.48	....	....	.010	434.46	.009	252.51	Forms, crossings, etc.
Cement	.400	8,395.47	.420	4,411.72	.402	18,401.57	.424	11,965.38	Delivered on job.
Gravel and sand, f. o. b. unloading point	.300	6,310.68	.275	2,889.82	.292	13,385.05	.076	2,137.98	Cost of material plus freight, if any.
Concrete surface labor	.307	6,419.50	.187	1,969.52	.258	11,775.94	.233	6,581.71	All labor on concrete including laying water pipe and operating pump
Steel for joints	.036	752.62	.034	352.55	.034	1,589.83	.036	1,007.51	Material only.
Felt for joints	.008	165.50	.008	81.50	.007	325.38	.009	258.64	Material only.
Sundries	.014	302.81	.012	121.19	.008	358.27	.003	94.79	Telephone, blacksmith, traveling expenses of foreman
Workmen's compensation losses and industrial insurance	.038	783.81	.031	320.55	.038	1,721.73	.036	1,007.31	Required by law in case of injury or lost time.
Machinery repair and depreciation	.040	828.83	.032	338.95	.040	1,820.60	.038	1,065.16	Prorated from total. Includes depreciation on pipe, at 33%.
Totals	$1.57	$33,498.78	$1.30	$13,699.55	$1.61	$73,583.62	$1.53	$13,050.64	
Length paved	11,805 ft		5,255 ft.		22,846 ft.		15,861 ft		
Width paved	16 ft		18 ft.		18 ft.		16 ft.		
Number sq. yds. (total)	20,798 sq. yds.		10,510 sq. yds.		45,692 sq. yds		28,197.3 sq. yds		
Average haul of material	2 miles		1 mile.		1½ miles.		2 miles.		

TABLE V. COST OF CONCRETE ROADS BUILT BY CONTRACT (1913).

	Name of Road.														
	Chicago. No.4.		Chicago No. 3.		Janesville No. 3		Janesville No. 2		Green Bay.		Mukwanago		*Kilbourn No. 3.		
Item.	Cost per sq. yd. paved.	Total cost.	Cost per sq. yd. paved.	Total cost.	Cost per sq. yd. paved.	Total cost.	Cost per sq. yd. paved.	Total cost.	Cost per sq. yd. paved.	Total cost.	Cost per sq. yd. paved.	Total cost.			Remarks.
Contract price (items below excepted)	$0.920	$15,206.68	$0.886	$8,138.77	$1.150	$19,595.20	$1.180	$30,512.91	$0.380	$5,969.65	....	$15.037.62		12,412.73	Includes all material and labor.
Shoulders	....	....	.012	144.75	.016	268.18	.001	294.17	.048	754.92	....	....	....	....	
Extras	001	12.96	.010	91.75	.011	241.00	.001	20.00	.057	895.70	....	....	....	....	
Coal, oil, etc., for mixer and pump	....	....	....	....	....	....	....	....	....	....	....	....	....	21.98	
Lumber, etc., for surfacing	....	....	....	....	....	....	....	....	....	....	....	99.36	.408	5,918.88	Net cost—Cement sacks were redeemed with less than 1½% loss.
Cement	.449	7,429.76	.440	4,069.48	.475	8,093.91	.471	12,166.87	.352	5,539.44	.425	5,819.21	....	....	
Gravel and sand f. o.b unloading point	....	....	....	....	....	....	....	....	.285	4,482.14	....	....	....	....	
Labor and inspection	.025	403.75	.033	301.17	.028	473.21	.007	178.29	.079	1,238.68	....	....	....	....	
Steel for joints	.032	530.94	....	....	.034	579.51	.035	898.78	.036	566.51	....	600.83	....	738.57	Material only.
Felt for joints	.008	132.51	.007	60.50	.008	142.52	.008	214.60	.009	133.10	....	170.00	....	175.60	Material only.
Sundries	..	5.14	.002	21.17	....	5.05	.003	63.35	.003	59.09	...	149.98	....	34.11	
Workmen's compensation losses and industrial insurance		.		....		....	....	....	....	....	....	98.81	...	....	Prorated from total for the year.
Machinery depreciation	...	....	..	....	....	....	....	....	....	....	....	104.48	....	....	Prorated from total for the year.
Totals	$1.44	$23,721.64	$1.38	$12,797.59	$1.73	$29,398.48	$1.72	$44,348.97	$1.25	$19,634.23	....	$22,080.29	....	19,251.87	
Length paved	8,264.5 ft.		3329 5 ft.		619-24 ft.		12,929.2 ft.		7,854.8 ft.		7,711.9 ft.		7,131.4 ft.		
Width paved	18 ft.		25 ft.		7,694.3-18 ft.		18 ft.		18 ft.		16 ft.		18 ft.		
Total number of sq. yds	16,529 sq. yds.		9,248.6 sq. yds		17,039.3 sq. yds.		25,858.4 sq. yds.		15,709.6 sq. yds.		13,710 sq yds.		14,262 sq. yds.		
Average haul of material	2/3 mile.		2/3 mile.		¾ mile.		3½ miles.		1½ miles.		1½ miles.		2½ miles.		

*Includes other items.

form subgrade for the pavement, also the completion of the grading some distance ahead and the more or less complete drainage obtained.

ABSTRACT OF SPECIFICATIONS FOR CONCRETE ROADS.

The following abstract from the specifications for this work give the detailed methods used.

Cement.—Some standard brand of Portland cement shall be used which has been in practical use on public work and shall have proved satisfactory therein. No brand of cement shall be used which the Engineer deems unfit for the work, nor shall any cement be used which fails to give satisfactory results according to the standard methods of testing as provided by the American Society for Testing Materials. The contractor shall provide sufficient means to protect the cement against dampness, and no cement shall be used which has become caked.

In case the county furnishes the cement delivered either F. O. B. cars or on the work, bags will be charged to the Contractor at the rate of 10 cts. each, which will be credited to him upon the return of the same, in acceptable condition, to the company furnishing the cement. The Contractor shall tie the sacks in bundles of 50 each in a manner approved by the Commissioner and shall secure from said cement company receipts for said bags, in addition to marking the number on both copies of the delivery tickets at the time said sacks are turned over to the cement company's teams. Each bundle shall be tagged in such a manner as will completely show the name of the contractor to whom credit should be given, as well as the name and number of the road. Any bags which may become lost, torn, hard or otherwise not acceptable to the receiving company shall be paid for by the Contractor. Any cement which may be delivered in good condition F. O. B. cars or on the work, and which may become lost or unfit for use through negligence of the Contractor or his employes, shall be paid for by the Contractor at the rate which is charged to the County by the company delivering said cement.

TABLE VI.—COST OF WOODEN GUARD RAIL (WISCONSIN STANDARD TYPE).

Item.	Total cost.	Cost per linear foot.
Length	8,637	.....
Labor	$1,472.05	$0.16
Lumber	1,129.18	.13
Paint and hardware	231.12	.03
Workmen's compensation (prorated)	69.63	.01
Equipment depreciation and repair (prorated)	73.64	.01
Totals (for single rail)	$2,975.62	$0.34

Contractor shall provide facilities on the job for properly protecting and keeping in good condition at least 1,000 bags of cement.

Coarse Aggregate. The coarse aggregate shall consist of clean, hard, sound flint, or other hard siliceous pebbles, having a reasonably uniform graduation from a size which will pass through a 1½-in. round screen to a size that is retained on a ¼-in. screen, and no gravel composed in part of slate, shale or disintegrated stone may be used. Crushed granite, quartzite, limestone, or trap rock, graded to the size specified above, may be used upon the approval of the engineer. In no case will coarse aggregate be approved which has a French coefficient of wear of less than 10, and none having any dimension greater than two inches will be allowed to be used. The right is reserved to reject for cause any or all stone or gravel delivered on the work.

Fine Aggregate.—The fine aggregate shall consist of clean, sharp grains of silica or hard silica rocks, and shall not contain over 3 per cent of clay or loam. The fine aggregate shall have a reasonably uniform graduation from a size which will pass through a ¼-in. screen down. Sand containing an appreciable percentage of flat flakes shall not be used. Fine aggregate other than the above may be used only upon the approval of the Engineer.

Pit Run Gravel.—Pit run gravel shall not be used except when screened to the above two sizes and delivered separately on the road. Aggregate containing lumps of clay or frozen material will not be accepted.

Proportions for Concrete.—The concrete for work provided for herein shall consist of 1 part of cement, 2 parts of fine aggregate, and 3½ parts of coarse aggregate, unless a larger proportion of coarse aggregate shall be ordered in writing by the Engineer, in which case a suitable deduction shall be made in the contract price if the Contractor provides the cement. The aggregate shall be placed in the mixer in such manner as to insure that a uniform amount of each class of aggregate is used in each batch of concrete; and the method of measuring the aggregate, whether in wheelbarrows or otherwise, shall be approved by the Commissioner. One sack of cement, 94 lbs. net weight, is to be considered as one cubic foot, and all measurements are to be by volume.

TABLE VII.—COST OF TARRING CONCRETE ROADS.

Item.	Total cost.	Cost per sq. yd.
Number of square yards	58,062	.....
Material for tarring	$2,989.61	$0.05
Labor	2,840.99	.049
Coal and sundries	72.54	.002
Workmen's compensation (prorated)	145.12	.002
Equipment depreciation and repair (prorated)	153.44	.003
Totals	$6,201.70	$0.106

The mixture above specified requires 1.68 bbls. of cement to the cubic yard of finished concrete. If the county furnishes the cement, the contractor will be allowed to use not more than 5 per cent in excess of this amount in any day's run, and if the above amount is exceeded by more than 5 per cent, the cost of the excess cement at its price delivered to the contractor shall be deducted from the contract sum. If a different proportion shall be ordered by the engineer, the cement content shall be changed accordingly, but the remainder of this paragraph shall remain in effect.

Roadbed.—The roadbed shall be considered as that portion of the road upon which the concrete is placed. The roadbed shall consist of the natural earth which has been brought to the proper cross section and rolled until firm and hard. When soft spots occur in the roadbed, they shall be dug out and the soft material replaced with good dirt or other material that will thoroughly compact. When the natural roadbed is so sandy that no compaction can be secured, the rolling may be omitted. Immediately before concrete is poured on the roadbed, it shall be carefully brought to the proper cross section and tamped until firm and even with a hand tamper weighing at least one-half pound to the square inch.

TABLE VIII.—SUMMARY OF COST.

Item.	Quantity.	Unit cost.	Total.
Earthwork (cu. yds.)	56,256	$0.76	$42,712
Guard rail (lin. ft.)	8,637	0.34	2,976
Tarring (sq. yds.)	58,062	0.106	6,202
Surface built by force account, avge. haul 1.6 miles (sq. yds.)	105,368	1.55	163,833
Surface built by contract, avge. haul 1.6 miles (sq. yds.)	112,358	1.53	171,233

Side Forms.—Before placing concrete, two-inch plank, free from warp and knot holes shall be placed on edge and securely staked in line with the outer edge of the pavement, the upper edge of the plank conforming to the finished grade of the edge of the road. Metal forms may be used upon the approval of the Commissioner. Wooden forms shall be thoroughly wetted, and metal forms oiled before depositing concrete against them. All mortar and dirt shall be removed from forms previously used before placing concrete.

Mixing Concrete.—Concrete shall be mixed with a batch mixer of a type approved by the Commissioner. The concrete materials shall receive at least five complete turns before being discharged, and if, in the opinion of the Commissioner, a greater amount of mixing is required, the above number of turns shall be increased until a thoroughly mixed concrete is secured. The aggregate shall be mixed with sufficient water to produce a concrete which can be spread into position with a template or straight edge, and handled without causing a separation of the coarse aggregate from the mortar. The amount of water used shall be increased or decreased from time to time as may be directed by the Commissioner.

Placing Concrete.—The concrete for the roadway shall be placed between the forms before described, the entire thickness of the concrete being placed at one time. When ordered by the Commissioner, the subgrade shall be wetted to his satisfaction before concrete is deposited. After the concrete has been deposited between the forms, it shall be spaded and tamped until the mortar flushes to the top, and the concrete shall be placed in such quantity that there shall be a slight excess between the forms. The surface of the roadway shall then be shaped to conform to the required cross section by striking off with a templet or strike board cut to the proper shape. This templet shall be drawn along the forms, and shall be held securely against the top of the forms, and shall be moved with a combined longitudinal and crosswise motion which will prevent dragging the larger particles of the aggregate and marring the surface. The templet shall be cut to crown for a distance at least one foot greater than the width of the concrete. When the strike board is within three (3) feet of a transverse joint it shall be lifted to the joint and the pavement struck by moving the strike board away from the joint; any excess concrete shall be removed. Concrete adjoining transverse joints shall be dense in character and any holes left by removing any device used in installing the joint or filler shall be immediately filled with concrete.

Concrete shall be handled rapidly into place and successive batches deposited in a continuous operation. Whenever an interval of more than 30 minutes shall elapse between batches, an expansion joint shall be provided and installed in the same manner as at the regular joints. A visible line of demarcation between successive batches shall be proof conclusive that the foregoing was not complied with, and the block in which the same occurs shall be rebuilt.

No concrete shall be deposited at a temperature of less than 40° F. unless all materials shall be heated and the concrete afterwards protected to the satisfaction of the Commissioner.

A Method of Determining Time of Performance of Work With Special Application to Grading.

The time factor in construction work is a variable which is dependent upon many conditions and is often of considerable importance. It appears that a practical method of approximating the number of working days to be allowed for a given contract is by carefully segregating, analyzing and averaging portions of similar work performed under approximately the same conditions.

In a paper before the Society of Municipal Engineers of the City of New York, Mr. G. L. Bennett, efficiency engineer for the New York Board of Estimate and Apportionment, presented a general discussion of this subject with a detailed method of handling grading work, which is in use by Mr. R. H. Gillespie, chief engineer of sewers and highways for the Borough of Bronx, New York. While the methods and costs given apply directly to New York City conditions, the method of approaching the problem is applicable to any locality.

Contracts commonly arise from demands which in themselves are either urgent and set for a particular time of fulfillment or are more complaisant as to time, requiring only ultimate completion within rather wide time limits.

Contracts of the former type, for emergency work or for supplying necessary links in larger schemes, can, in proportion to their needs, afford to sacrifice economy for dispatch. Contracts of the latter type, which includes a majority of street improvement work, can properly afford to disregard time as such and to seek economy of total costs alone.

Definition of Contractor's Economical Time.—There are comparatively few pieces of work which can only be economically accomplished by the use of some one particular equipment and of some one particular method. In general, there are a number of equipments and methods which, depending upon the genius of those in control for management under the conditions obtained will yield economical results, but will require somewhat different times for completion. Leaving out of account variations in required time due to such causes there is for each sort of work some number of working days, more than which could not economically be used by the contractor. Thus, on a pick and shovel job, the employment of less than a certain number of men would not be economical because of the cost of the foreman and superintendence; similarly, the use of more than a certain number of working days by a contractor would be uneconomical. The same thing applies to any job for which an equipment and force are provided sufficient to complete that job in one of the perhaps several most economical ways so far as contractor's costs are concerned; and to this somewhat varying time may be applied the term, "Contractor's Economical Time."

Equipment Warranted.—It is to be recognized that small total contract quantities, in general, only warrant the employment of light and easly moved equipments and that as the quantities become larger more and still more effective equipments are warranted.

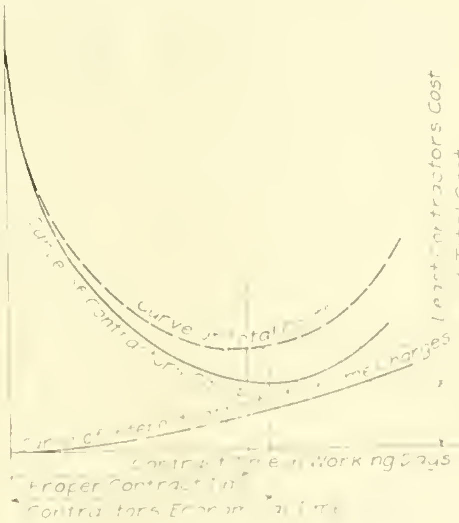


Fig. 1. Curves Showing the Relation of Total Cost to Contractor's Economical Cost.

But this may be modified considerably by the amount of work of this nature done or yet to be done, in the locality of the contract in question. Thus a rather small contract for rock excavation could properly be given a shorter contract time in the northern part of Greater New York where rock excavation is constantly in progress than it could somewhere out on the shores of Long Island or southern New Jersey.

Relation of Labor Cost to Equipment.—The amount of equipment to be employed is seriously effected by the costs of labor and the ease of procuring equipment. Where satisfactory labor is expensive or difficult to procure, contractors will, in general, employ machinery of a type which otherwise would only be used on much larger contracts, resulting, of course, in a shorter contractor's economical time. Where machinery can be easily hired, equipment will often be used on small jobs such as could not otherwise be afforded. This affects also very large jobs for which, where no satisfactory disposal can be made of worn machinery, equipment is often provided only in such quantities that it shall all be practically worn out when the job is completed.

Definition of Total Cost.—The total cost of the work rather than the cost to the contractor is the matter which interests the engineer in his capacity of manager for the party contracting for the work. And the total cost is, of course, the contract price plus the costs of surveys and designs, plus the costs of inspection, superintendence and interest on the moneys invested by partial payments or otherwise, all of which latter vary nearly directly with the time taken for the work.

Two Methods of Determining Time.—Two methods of ascertaining the time to be allowed were open:

(a) By the balancing of inspection, superintendence, interest and similar time charges against the increased costs of obtaining and operating equipments of more capacity than are required to complete the contract in the contractor's economical time. That point on the curve which gives the least total cost, is the correct time to allow for the contract.

(b) By plotting the times allowed on previously completed contracts composed mainly of one kind of work and which had, in the judgment of the engineers in charge thereof, been prosecuted vigorously and with adequate equipment, a series of curves of quantity with reference to time can be drawn for each kind of work, each curve recognizing in its equation some particular controlling factor of variation. Having such curves and knowing the total quantity of work to be done, the proper contract time can be ascertained and the results combined to give the time for a contract including various kinds of work.

METHOD A.

The operation of the first method for arriving at the contract time is shown diagrammatically in Fig. 1.

In Fig. 1 the curve of contractor's costs shows the variation in cost with the contract time; the curve of interest, inspection and other time charges, the variation of these with contract time. The curve of total costs which has for its ordinate at each point the sum of the ordinates of the other two curves at that point, shows the variation of both with the contract time.

The cost of interest, inspection, etc., will decrease therefore with a decrease in contract time, but because of the greater cost of equipment, etc., necessary to complete the work in less than a contractor's economical time, the contract price will tend to increase. That time which will give the minimum total costs for the work which should accordingly on this sort of contract be used as the "contract time," will therefore be somewhat shorter than the "contractor's economical time."

As these curves do not show variations with regard to quantity or other conditions governing any one sort of work, one such set of curves is necessary to determine the proper contract time for each total quantity of each kind of work. The contract times can later be plotted against the total quantities as has been done for Method B, shown in Figs. 2, 3 and 4.

It is to be noted that this method depends upon a curve of contractor's costs, which can be determined, point by point, for the total quantity and the kind of work to which this curve applies, only by designing the most efficient plant or equipment to do this amount of this kind of work in each of the varying times assumed and from the use and costs of these plants or equipments, arriving at the contractor's costs.

There is a very considerable difference in judgment as to the most economical plant for any given contract time and total quantity of work to be done and still more difference as to the delays and other items summed up in the contractor's costs. This is apparent to those who are familiar with the variations in bid prices on works of such magnitude as call for new and specially designed equipments.

And that the very considerable variation in bid prices between the various bids received for any contract is not all due to differences in the profits which the different bidders desire, is quite clear to those who have studied prices bid when work was scarce and when, therefore, most if not all the bidders were competing keenly for the job.

That there is ample room for considerable differences between even the most carefully estimated contractor's costs and the actual costs is quite evident to those who have planned and estimated and later built and operated contractor's plants.

METHOD B.

In view of the large number of assumptions which must be made and the work involved the first method has been discarded in favor of the second.

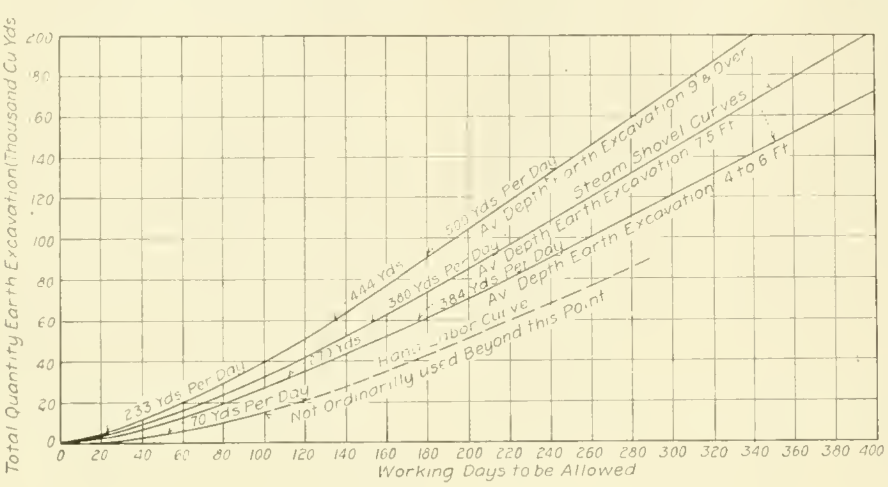


Fig. 2. Curve for Earth Excavation.

RULES FOR INTERPRETING CURVES USED IN METHOD B.

Determination of Contract Time.—To determine the number of days which shall be written into any contract for regulating and grading as the number of working days to be allowed, the curves hereto attached are to be used in accordance with the rules herein given.

Explanation.—The work to be done will consist of items of rock or earth excavation or of filling or of both, of curb, of flagging, and of bridge-stones. These may or may not be accompanied by walls of dry rubble, rubble in mortar, or concrete; by pipe drains; by inlets; by receiving basins; by manholes; by piles; and by special constructions.

Items Which Govern.—Ordinarily, work on more than one of these items can be prosecuted at the same time. Care is therefore required to so use the information available of the special conditions surrounding a proposed work as to eliminate from consideration all of those items and all parts of items which can properly be done during the progress of some other item. There will thus be left as the determining factor in the required time for such work one, or, more rarely, two main items which can not be done simultaneously and some preliminary and subsequent parts of items.

The main items are liable to be earth and rock excavation or filling. The preliminary parts of items: the excavation of sufficient earth to permit earth and rock excavation to progress simultaneously; the building of sufficient of the pipe drains, inlets, manholes or basins to permit such building and the filling to progress simultaneously or the building of sufficient walls, in those few cases where such wall is necessary before any filling can be done, to permit such wall building and filling to progress simultaneously. The subsequent parts of items: special structures which can not be completed until after the main item is completed; curb, flagging or bridgestone.

The curves have been drawn to recognize, in view of the total quantity of an item to be done, the equipment and force which should be used.

They show, accordingly each for its condition, the amount of time for any total quantity which should be consumed in completing that quantity.

Rule 1.—If the time for a part of an item has to be estimated, it is to be taken therefore at the same rate of accomplishment per day as is the total quantity.

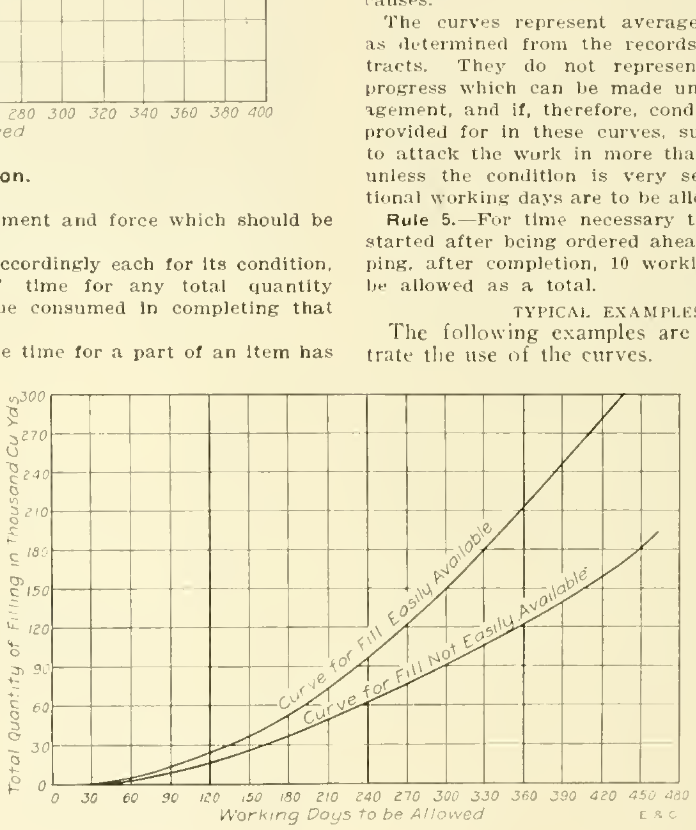


Fig. 4. Curve for Filling.

Rule 2.—Where conditions are clearly intermediate between those shown by the curves, interpolation is permissible, but where doubt exists it is preferable as making for lower costs to take that nearest diagrammed condition which gives the longer contract time.

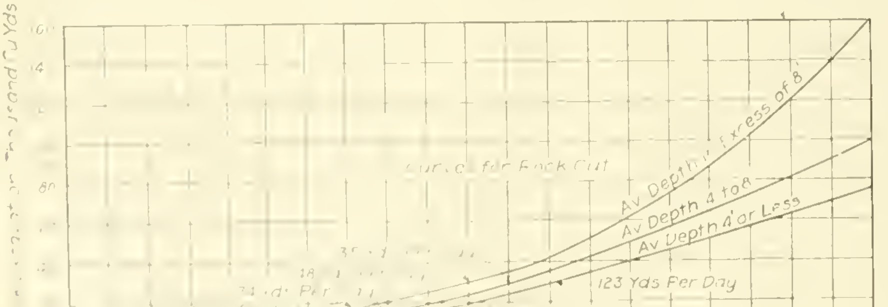


Fig. 3. Curve for Rock Excavation.

Rule 3.—Where a part of an item comes clearly under one condition recognized in the curves and the remainder as clearly under another, unless the equipment and force which should properly be used for doing these two parts is widely different, the time for the two parts, each taken at the same rate of accomplishment as if the total quantity came under that part's condition, shall be summed to give the time for that item.

Rule 4.—If the equipment, etc., should properly be different, the time for each part of the item is to be taken as if the quantity for this part item were a total quantity and the times so obtained, summed to give the total time for the item.

Note.—It is to be remembered that the contract provides that allowances of time for delays occasioned by the weather, or by any act or omission on the city's part, are to be made in addition to the number of working days. No consideration need be given in their determination to any conditions arising from such causes.

The curves represent average good practice as determined from the records of many contracts. They do not represent the greatest progress which can be made under good management, and if, therefore, conditions arise not provided for in these curves, such as inability to attack the work in more than a few points, unless the condition is very severe, no additional working days are to be allowed as a total.

Rule 5.—For time necessary to get the work started after being ordered ahead and for stopping, after completion, 10 working days are to be allowed as a total.

TYPICAL EXAMPLES.

The following examples are given to illustrate the use of the curves.

Example 1.—On a contract with a center line length of 8,900 ft. and a street width of 100 ft. there are the following items and quantities:

Earth excavation	88,000 cu. yds.
Rock excavation	26,700 cu. yds.
Fill	151,100 cu. yds.
Dry rubble masonry	700 cu. yds.
Rubble in mortar	25 cu. yds.
12-in. pipe	100 lin. ft.
18-in. pipe	575 lin. ft.
Manholes	4
Guard rail	5,800 lin. ft.
Lumber	7,500 ft. (B. M.)

Filling.—It is known that over a portion of the work where filling is required the street is located on a swamp where settlement will, in all probability, take place. Assume that this settlement will amount to 30,000 cu. yds. Then the total filling required to complete the work would be 151,100 + 30,000 = 181,100 cu. yds.

The sum of the earth and rock excavation amounts to 114,700 cu. yds. It should be assumed that the entire excavation is to be applied to making the fill so the material can be considered as easily available.

The balance of the material required for filling (66,400 cu. yds.) must be obtained from outside sources. It is further known that the swamp section of the street is near tide water where material can be obtained by scows. This material, so obtained, should be classed as material "easily available." Even though the dock or nearest obtainable landing may be at some distance from the street under consideration, and especially in view of the possibility of obtaining and placing this filling during the progress of the grading on other portions of the work, it should be classed as "easily available."

An examination of the filling diagram will therefore indicate that considering 181,100 cu. yds. as "easily available," 332 days should be allowed, and adding to this 10 days for starting and stopping, we have 342 days, or say 345 days for the contract time.

Excavation.—If, on the other hand, we consider the excavation and know that the earth cutting averages from 4 to 6 ft. in depth, and the rock 4 to 8 ft. in depth, and that 10,000 cu. yds. of earth must be excavated before rock excavation can begin, and that thereafter both will be carried on simultaneously, we will obtain from the curves the following:

10,000 cu. yds. earth excavation (at 88,000 rate)	27 days
26,700 cu. yds. rock excavation	212 days
	239 days
Starting and stopping	10 days
	249 days

If we consider only the earth excavation, and assume that while same is in progress the rock will be excavated, we have from curves the following:

88,000 cu. yds. earth excavation	235 days
Starting and stopping	10 days
Total time required	245 days

It is evident from the above that the filling required on the work controls and that the contract time should be fixed at 345 days.

Example 2—

Earth excavation	1,000 cu. yds.
Rock excavation	500 cu. yds.
Filling	120,000 cu. yds.

In this example the excavation is plainly not to be considered. The filling if easily available will by the curves require 268 days. If not easily available, filling will require 357 days.

To either of these 10 days should be added for starting and stopping, making either 278 days which call 280 days, or 367 days which call 370 days.

Example 3—

Earth excavation	6,000 cu. yds.
Rock excavation	6,000 cu. yds.
Filling	12,000 cu. yds.

If earth overlays rock, the quantity of earth which must be removed to permit rock and earth excavation to progress simultaneously, must be determined from a knowledge of the local conditions. If these conditions show that say, 35 per cent of the earth has to be removed before the rock excavation can be properly commenced, and that the rock has an average depth of four feet or less, the times required for excavation will be:

For earth 35 per cent of the 64 days required by curve for hand labor for 6,000 yds.

For rock 114 days required by curve for 6,000 yds. of average depth four feet or less.

The sum of these two, plus 10 days for starting and stopping equals 146 days which call 150 days.

The filling, which is all easily available, would only require 90 days.

Therefore the contract time for this job would be 150 days.

If earth and rock are in separate cuts and separately approachable so that the two sorts of excavation can properly progress simultaneously, the earth excavation need not be considered. The filling will, of course, not be the determining factor and the rock excavation will be. Under these conditions, the contract time should be for rock excavation, 114 days plus 10 days for starting and stopping equals 124 days, say 125 days.

Example 4—

Earth excavation	20,000 cu. yds.
Rock excavation	2,000 cu. yds.
Filling	3,000 cu. yds.

In this case, the earth overlies subgrade rock throughout most of the work.

The rock excavation remaining to be finished after the earth excavation is completed will amount to about 18 per cent of the total rock excavation.

The filling will not determine the required time.

The earth excavation being all in shallow cut will be taken out by hand labor, thus requiring 118 days.

The 2,000 yd. rock, 4 ft. cut or less, would require 36 days, and 18 per cent of this would require 6 days.

The sum of the above plus 10 days start and stop equals 134 days which call 135 days.

Example 5—

Earth excavation	1,000 cu. yds.
Rock excavation	6,000 cu. yds.
Filling	7,500 cu. yds.

Here, the 1,000 cu. yds. earth overlies the 6,000 cu. yds. rock and the conditions show that only a very little earth will be taken off, say, 200 yds. before the rock is commenced. Two hundred yards earth should require about one-fifth of the 24 days required by hand labor curve for the total 1,000 yds., say 5 days.

Six thousand cubic yards rock, of a four foot or less depth of cut, requires by the curve 114 days.

The sum of the above plus 10 days for start and stopping, equals 129 days which call 130 days, contract time.

Example 6—

Earth excavation	30,000 cu. yds.
Rock excavation	15,000 cu. yds.
Filling (easily available)	120,000 cu. yds.

The filling will require by the curve 268 days.

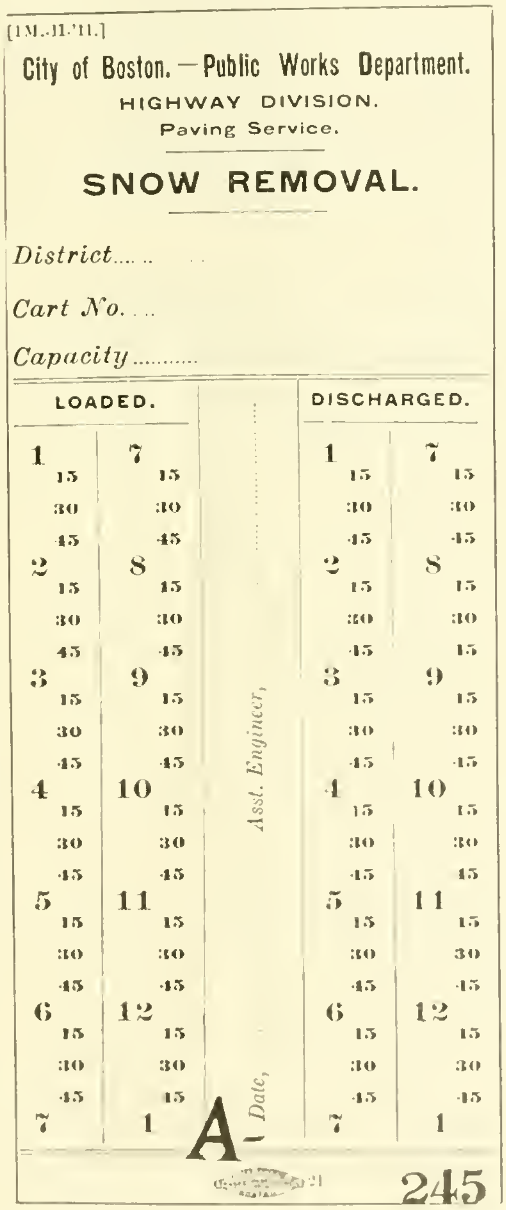
The earth and rock excavation are such that they can be prosecuted quite simultaneously. The rock excavation will therefore control and has an average depth of 6 ft., requiring by the curve 270 days, so that 270 days is good for either.

Therefore 270 plus 10 days stop and start gives 280 days as the contract time.

The application of this method to others of the more usual types of municipal work is obvious and in some cases is under way.

The confidence of contractors in general in the absolute fairness and in the knowledge of the engineer, will, perhaps as much as any other factor, tend to lower the costs of work to be done. And to this end a uniform method of figuring the contract time rather than guessing at it, will, it is believed, contribute in no small degree.

It is probable that the curves herein given will require modification in some instances to make them fit to the experience of other localities, and it is possible that a further study of data similar to that upon which these curves are based may lead to somewhat higher or lower averages of performance, resulting in shorter or longer contract time.



[1M.-11-'11.]

City of Boston.—Public Works Department.

HIGHWAY DIVISION.

Paving Service.

SNOW REMOVAL.

District......

Cart No....

Capacity..........

LOADED.		DISCHARGED.	
1	7	1	7
15	15	15	15
30	30	30	30
45	45	45	45
2	8	2	8
15	15	15	15
30	30	30	30
45	45	45	45
3	9	3	9
15	15	15	15
30	30	30	30
45	45	45	45
4	10	4	10
15	15	15	15
30	30	30	30
45	45	45	45
5	11	5	11
15	15	15	15
30	30	30	30
45	45	45	45
6	12	6	12
15	15	15	15
30	30	30	30
45	45	45	45
7	1	7	1

Asst. Engineer,

Date,

A-

245

Fig. 1. Driver's Time Ticket, Snow Removal, Boston, Mass.

Methods of Handling Snow in Boston, Mass.

Snow removed in Boston, Mass., is a somewhat different problem than it is in most cities because the narrow streets prevent piling; the snow has to be removed as fast as it is collected. The organization and methods employed are outlined in a paper presented before the Philadelphia Snow Removal Conference by Mr. L. K. Rourke, commissioner of public works. An abstract of this paper follows:

Traffic has not been blocked by snow for many years in the city of Boston. An ordinance provides that the abutters must remove the snow from the sidewalks; this they usually do by shoveling it into the gutters. The city forces take care of the snow between curbs, except on streets where the surface car tracks are located.

Divisions of Work.—The work in Boston is divided into two parts; first, the handling of snow in the suburban districts on about 470 miles of streets; second, the removal of snow in the city proper from about 100 miles of streets.

The work in the suburbs is done by the regular city forces and consists of ploughing out the gutters, cleaning crosswalks and, when the snow is deep enough to warrant it, breaking out the street with a breaker or leveler usually drawn by six horses.

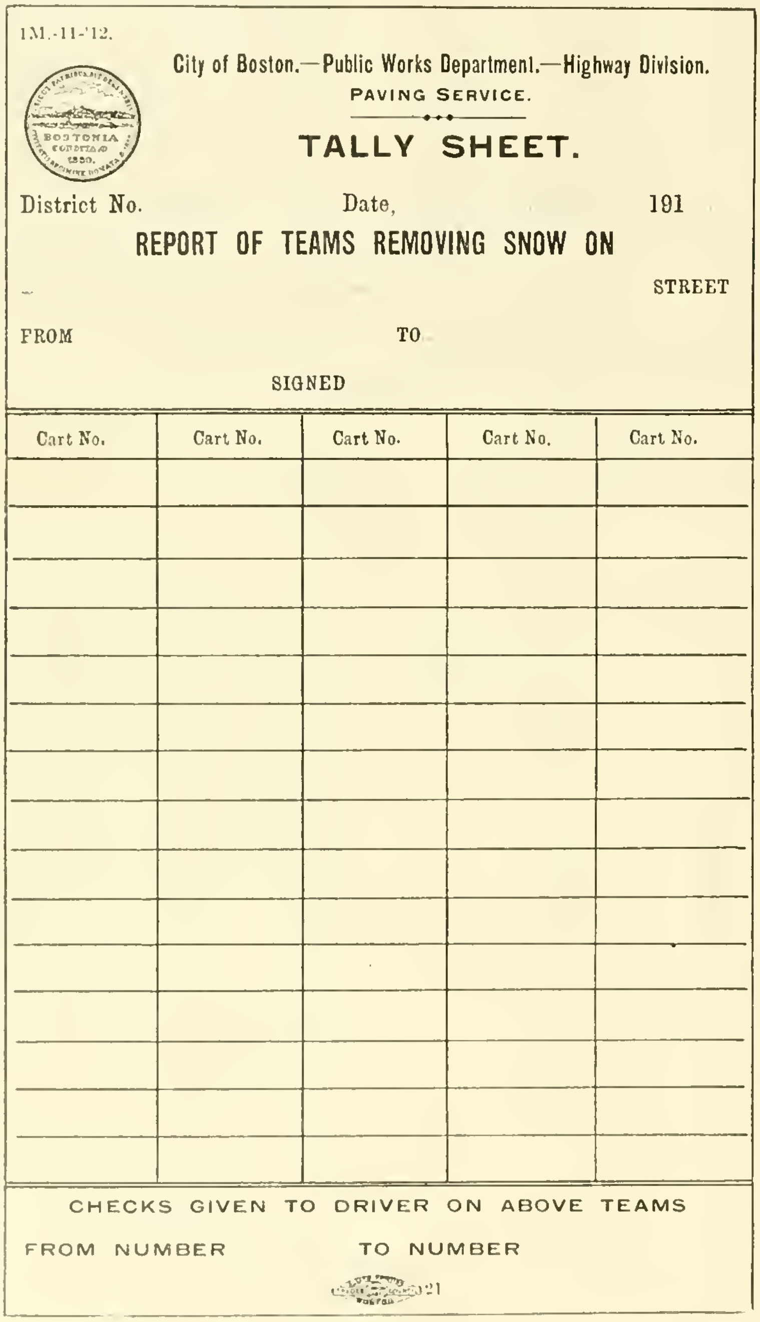
The task in the city proper is an entirely different proposition. The business streets of Boston are so narrow that there is just one thing to do with the snow, namely, to remove it once for all; there is no such thing as piling it up to wait for a thaw. In this connection Boston is very fortunate, in that her great water front furnishes plenty of dumping places with a short haul. To a limited extent, snow is also dumped into certain sewers.

Organization.—In handling what might be termed an ordinary storm, up to 6 or 8 ins. of fall, the regular city forces are sufficient, consisting of about 1,000 men and 300 carts. With a light snow, machine sweepers are used to sweep the snow to the gutters, where it is loaded into carts by hand. As the snow becomes heavier on the smooth paved streets ordinary road scrapers are used to pile the snow for the shovelers. To take care of heavier storms, such, for instance, as that of last February, when we had a fall of 23 ins., the regular city force is inadequate and extra help is provided for, as follows:

The city proper is divided into six snow districts, and every fall bids are called for, by advertisement, for the removal of snow by the cubic yard, cart measurement, in these districts. The contracts are awarded to the lowest bidder. No amount of work is guaranteed the contractor; in fact, some years he receives no work.

Equipment.—The equipment of the contractors is varied and comprises single and double dump carts, pungs and motor trucks, all loaded by hand. The contractor is required to coat the inside of his carts with heavy oil or grease. The carts are classified according to their capacities, as follows:

A. Any cart, wagon or truck holding not less than 1 cu. yd. and not 2 cu. yds. (water measure in both cases) shall be deemed to hold only 1 cu. yd. and be classified as Class A.



1M.-11-'12.

City of Boston.—Public Works Department.—Highway Division.

PAVING SERVICE.

TALLY SHEET.

District No. Date, 191

REPORT OF TEAMS REMOVING SNOW ON

STREET

FROM TO

SIGNED

Cart No.	Cart No.	Cart No.	Cart No.	Cart No.

CHECKS GIVEN TO DRIVER ON ABOVE TEAMS

FROM NUMBER TO NUMBER

Fig. 2. Tally Sheet for Loading and Dumping Foreman, Snow Removal, Boston, Mass.

B. Any cart, wagon or truck holding not less than 2 cu. yds. and not 2½ cu. yds. (water measure in both cases) shall be deemed to hold only 2 cu. yds. and be classified as Class B.

C. Any cart, wagon or truck holding not less than 2½ cu. yds. and not 3 cu. yds. (water measure in both cases) shall be deemed to hold only 2½ cu. yds. and be classified as Class C.

D. Any cart, wagon or truck holding not less than 3 cu. yds. and not 3½ cu. yds. (water measure in both cases) shall be deemed to hold only 3 cu. yds. and shall be classified as Class D.

E. Any cart, wagon or truck holding not less than 3½ cu. yds. and not 4 cu. yds. (water measure in both cases) shall be deemed to hold only 3½ cu. yds. and shall be classified as Class E.

F. Vehicles with a capacity of 4 cu. yds. or over (water measure) will be subject to special measurements in each case. Measurements will be made under direction of the Commissioner in the presence of a contractor's representative, and the capacity will be permanently marked upon the vehicle in a manner satisfactory to the Commissioner.

Any vehicle once classified and ticketed shall remain so for 12 hours; i. e., no change shall be made in a vehicle which will increase or diminish its capacity during that time.

Each vehicle must be provided with a sign or signs of such form, size and material as shall be required by the commissioner. The signs must be permanently and firmly attached to the trucks on the right-hand side of the vehicle, near the front end, in a manner satisfactory to the commissioner, and shall plainly show the contractor's name, the registered number of the truck, the class and the registered capacity. No vehicle will be registered until the signs have been permanently attached and no account will be taken of snow hauled in vehicles which have not the proper signs, and should it be found at any time that a vehicle is used with a sign or signs upon it to which it is not entitled, the latter must be replaced with the proper sign or signs, and any previous tickets issued on account of snow hauled in said vehicle must be corrected to conform to the facts, and any excess allowance of snow which may have been already made shall be deducted.

After the teams are measured and classified, a ticket, Fig. 1, is given each driver. This ticket is kept by the driver during the day or night, as it may happen to be, and at the close of the period he is supposed to turn the ticket over to his employer, the contractor. On delivering his bill for payment the contractor must accompany the same with these official tickets. No payment is made for tickets lost.

Method of Conducting Work.—The commissioner stations at each of the places designated by him for loading snow and ice, a loading foreman, or other representative, whose duty shall be to punch the official tickets presented by the drivers of the vehicles engaged in the removal of snow and ice. These tickets are to be punched at the nearest division of time as printed on the said tickets and only when the vehicle is fully loaded at the place designated and the ticket is presented by the driver himself with the vehicle containing the snow and ice. These tickets are to have entered upon them in indelible ink a series number of the vehicle, together with its letter of classification and the capacity. The tickets are subject to comparison at any time during the continuance of the contract with the register in the central office of the Public Works Department, and if at any time discrepancies are found to exist between them, the figures of the register are to be taken as correct, subject, however to an official measurement if requested by the contractor.

At each of the designated dumping points the commissioner shall station a dump foreman, or other representative, whose duty shall be to punch the official tickets at the nearest division of time as printed on said tickets of those drivers who have discharged full-sized loads of snow and ice at the proper place.

As an additional check on this system, a tally sheet, Fig. 2, is kept by both the loading and dumping foremen. As each ticket is punched by the loading inspector, he enters on this sheet the number of the cart and the time the ticket was punched. A similar entry is made by the dumping inspector. These sheets are sent to the main office at the end of each period of work and there placed on file.

In the removal of snow the street railway assists materially, although there is nothing in its franchise which compels this. In order to run its cars, the company removes the snow from its tracks by ploughs, which leaves the snow piled up in a ridge. A city ordinance forbids the depositing of snow in any street unless it is left in a safe condition. For the railway to comply with this ordinance, it is necessary for it to remove the snow ridge left by its ploughs. This is done by loading the snow into carts and pungs and hauling to the nearest dumps. In addition, at night, the railway runs gondola cars for hauling away the snow. These cars are hauled to one of our numerous bridges where the snow is unloaded into the water. This method of removing snow by cars is the most efficient of any in vogue.

In conclusion, the main point in coping with a snow storm effectually is to tackle it early and keep up with it.

Method of Staking Out a Bridge Site.—The following is a method of staking out a bridge site recommended by Mr. A. R. Hirst in his instructions to superintendents:

Where it is decided to stake out a bridge, the center line should first be established by fixing center points at some distance away, preferably at the limit of sight in each direction. Permanent stakes should then be lined by sighting between these, one near each end of the bridge, and a nail be driven in each. All later measurements should be from this line. The footings should first be laid out with reference to the center line. After this the neat work of the abutments also from the center line, and lastly, the superstructure.

In this way a mistake in the beginning of the work will not be continued in the later work, and even if the abutments should be built slightly out of line, the superstructure can be properly placed, so as to be in line.

Where grade is shown on drawings it is usually by a spike driven in a nearby tree or telephone pole. Where the commissioner does his own staking, this method of marking the final grade should be used.

Sewerage

Experts' Preliminary Report on Future Water Supply and Sewage Disposal Policy of Chicago.

An article on the present status of water supply and sewage disposal conditions in Chicago published in the sewerage section of this journal for April 15, 1914, announced the employment by the Chicago Real Estate Board of a board of experts, not previously interested in Chicago sanitary matters, to make a study of these conditions and to recommend a future sanitary policy for the city. The board of experts, composed of George A. Soper of New York City, John D. Watson of Birmingham, England, and Arthur J. Martin of London, England, met in Chicago on April 18 and studied the situation at close range for a period of two weeks. Their preliminary report, dated May 1, is here presented in substance. The studies were made by an actual field inspection of conditions, the board following the course of the sewage along the Chicago River, the Main Drainage channel, the Desplaines and Illinois Rivers for a total distance of 128 miles, and by the study of recorded data freely supplied by various officials, engineers and others. The possibilities of various methods of disposing of the sewage, other than by the present plan, were considered in the light of available data. The board of experts confined themselves to the large and essential aspects of the problems before them and did not enter into minor details. The following are excerpts from the report:

EXISTING PLAN OF DISPOSAL.

The plan of main drainage and sewage disposal in Chicago is based on the assumption that Lake Michigan forms the only source of drinking water available and that the purity of the water can be insured if the sewage is prevented from flowing directly or indirectly into the lake.

The works built to dispose of the sewage have been based on the theory that it would be permissible to discharge the crude sewage of the city into the Chicago River and flush it with a certain amount of diluting water from the lake, by means of the Sanitary Drainage and Ship Canal, into the Desplaines River system, which ultimately empties into the Gulf of Mexico.

It appears that the theory underlying the plan of disposal which is in force was decided upon before the works were constructed and the organic law providing for the creation of the Sanitary District specified the allowance of water which was to be provided for diluting purposes. It has been stated since the canal was built that the ratio of sewage to water required for the disposal of the sewage represented a minimum, not an average, or maximum, figure; and that it referred to the sewage of a mixed residence, and business territory from which unusual amounts of troublesome trade wastes were excluded. The accuracy or otherwise of the statements has little bearing on the situation. The conditions of sewage disposal which exist in the Chicago River and its outlet are conceded to be unsatisfactory, and it is our opinion that the disposal of the sewage cannot be made to answer the reasonable demands of sanitation without radical improvement.

PRESENT QUALITY OF THE WATER SUPPLY.

The vital statistics of the city apparently show that the present method of sewage disposal abundantly protects the drinking water against infection. The lowered death rate and the typhoid death rate have been taken to indicate the measure of this protection. It appears that Chicago has one of the cleanest and purest waters of any large city in America. It seems unnecessary to review the evidence or inquire whether all the improvement in the death rate and typhoid rate is to be ascribed to the water supply. It is sufficient to remark that there seems generally to have been sufficient protection afforded up to the present time.

The method of disposing of the sewage is not the only protection which has been taken against the transmission of disease by means of the drinking water. The intake cribs, of which there are six, have been extended further and further out into the lake, until one is about four miles from shore. If the reversal of the flow of the Chicago River is complete and no sewage is discharged into the lake, it is difficult to understand the necessity for placing the intakes so far from the land. Again, it is not apparent why one of the intakes should be four miles from shore and the others between two and three miles from the land. Apparently the object in placing the intakes out as far as practicable into the lake has been to avoid not only contamination along the water front, but to get water which is clear. At times of storm the water of Lake Michigan becomes turbid near shore, clay being conspicuously present. After fresh easterly winds even the best water obtainable from the four-mile crib becomes noticeably turbid.

As a means of protecting the water supply against occasional and accidental pollution from vesesls, which may have occasion to pass near the intake cribs, steps have been taken to place tanks upon the boats to receive their sewage. Arrangements have been made at some of the intakes for the application of hypochlorite as a temporary or permanent procedure. Statistics show that those parts of the city supplied with this disinfected water have been particularly free from typhoid fever.

CONDITION OF THE DESPLAINES AND ILLINOIS RIVERS.

The condition of the Desplaines and Illinois Rivers with respect to pollution produced a strong impression on the experts. On the days when the inspections were made the water was offensively polluted with sewage as far as Ottawa, on the Illinois River, 85 miles from Lake Michigan. Where bridges crossed the stream it was possible to see unmistakable evidences of the sewage. Odors were noticeable, grease lay upon the surface of the river, dead fish were numerous, and large quantities of sewage fungus were observable in the water. Disagreeable odors were noticeable at a distance of nearly one-quarter of a mile from the Marseilles dam. Above this point the board is informed that animal and vegetable forms which are natural to a normal river are absent. Through the whole course of the stream from Chicago to and below Ottawa, gulls were seen feeding on the solid refuse which the sewage brought down. That Joliet and other municipalities along this water course discharged their sewage and industrial wastes into the water, and that the condition of the stream is now better than it was in former years, does not excuse Chicago for discharging the sewage which forms the great burden of polluting matter observed.

OPINIONS AND SUGGESTIONS

To the foregoing brief statement of the situation as it is now regarded by the board of experts the following opinions and suggestions are added:

Dilution alone cannot be relied on as a permanently satisfactory method of disposing of the sewage of the district. It offers an insufficient means of disposing of the sewage at the present time, and would prove still more inadequate in the future, in spite of the full quantity of water which the Drainage Channel can carry. It will ultimately be necessary to treat practically all the sewage before it is discharged.

The procuring of a satisfactory water supply by diverting the sewage to the Drainage Channel and extending, or altering, the location of the intakes is not capable of furnishing a drinking water of suitable quality. For the protection and improvement of the supply the city should look forward to the construction of filtration works.

The conditions of sewage disposal and water supply which now exist are not so insanitary as to call for improvements which will involve immediate and large expenditures of money. Conditions which are insanitary in the extreme exist in places, but they can be corrected without excessive cost. The great need is for a definite and carefully worked out sanitary policy for the future.

The disposal of the sewage by discharging it into the Drainage Channel after suitable preparation, the protection of the water supply by diverting the sewage from the lake, and the maintenance of conditions in the Chicago River suitable for navigation, are closely connected subjects and the provisions to be made in future for sewage disposal, water supply and shipping should be worked out with this fact clearly in mind. The preparation of comprehensive plans for sewage disposal, water supply and river development should be undertaken immediately.

It is practicable at this time to enunciate certain principles and lines upon which it may be desirable to proceed. The suggestions follow:

1. The principle of diverting the sewage from the lake should be adhered to, in spite of any degree of sewage purification or water purification which may be employed.

2. Full advantage should be taken of as large a volume of diluting water as may be obtainable from the lake, in order that the cost of finally disposing of the sewage may be reduced to a minimum.

3. Thorough and exhaustive study of the capacity of the drainage channel to absorb and oxidize sewage should be made in order that full advantage may be taken of the opportunities thereby afforded for completing whatever process of sewage treatment may be employed as a preliminary measure of disposal.

4. It is desirable to concentrate as much of the sewage as practicable at a point beyond the built-up sections of the city and in the vicinity of the Drainage Channel, where the sewage can be treated sufficiently to permit the effluent to be discharged without serious danger of offense.

5. To avoid excessive cost in the construction of intercepting and collecting sewers, and in order to relieve the Drainage Channel as far as practicable, it will be desirable to treat the sewage of some parts of the city in the areas in which the sewage is produced, and not carry it to the central disposal station.

6. Practically no sewage should be discharged into the Chicago River nor the Drainage Channel, without some form of treatment, except in times of storm, when suitably located overflows will be permissible. The sewage should, for the most part, be collected by a system of intercepting sewers running along the water front and extending to the central disposal station.

7. The minimum requirement in the treatment of the sewage at the main station should be the removal of the suspended solids. Screens and settling basins now afford the most practical means by which the suspended matters can be removed.

8. Screens and grit chambers afford the most effective means of treatment which it is permissible to employ in the closely built up sections of the city. Owing to the odors that occasionally may be produced, and because of the antagonistic popular opinion which would, in all probability, be aroused, it will not be advisable to locate settling basins or more efficient processes of sewage treatment in the built up sections of the city.

9. With the growth of population and the development of the public opinion it is probable that a greater purification of the sewage will be called for in time than is possible by screening and sedimentation. This probability should be anticipated as far as possible and

the work so designed as to permit of extension and improvement without excessive cost.

10. Sanitary standards of cleanness for the water should be prepared to serve as a guide toward which the work of disposing of the sewage and protecting and improving the drinking water should tend. The standards should be interpreted liberally and with due regard to the modifying circumstances imposed by local conditions.

11. Different standards for different places should be prepared, including one for the lake water in the vicinity of the intakes, another for the Chicago River and its immediate tributaries, and a third for the Drainage Channel at its discharge into the Desplaines River.

12. The standard for the drinking water should contain specifications as to its bacterial condition and turbidity; that relating to the river should have reference to the appearance of the water and its odor, and that which refers to the discharge of the Drainage Channel to the suspended solids and putrescibility.

13. In regard to the condition of the water discharged from the Drainage Channel into the Desplaines River, the ultimate end should be a discharge which will not render the water of the river offensive or materially more difficult for the municipalities upon it to make, and keep, clean.

14. The arms of the Chicago River which have no present or future value for navigation, and which are now in a foul and stagnant condition, should be filled in.

15. Pending the construction of the intercepting sewers and sewage treatment work, the Drainage Channel and River should be kept free from putrefying deposits by dredging and the dredgings should be disposed of otherwise than by dumping into or near the lake.

16. The sites for the treatment works should be determined upon as soon as possible, and sufficient land should be reserved to meet the future requirements which can be foreseen and afford a liberal margin for contingencies.

17. All considerable flows of especially foul liquids, such as those from the stock yards, should receive special treatment before they are discharged into the Chicago River or Drainage Channel. The cost of treating these wastes, above the cost of treating the same volume of sewage from the rest of the city, should be borne by those who produce the wastes.

18. The disposal of the sewage in the Calumet territory is a local problem which can and should be solved by the construction of one or more purification plants.

Colliers for Panama Canal.—Contract for building two colliers, equipped ready for service, has been let by the Panama Canal Commission to the Maryland Steel Co. at the price of $987,500 each. It is expected that they will be placed in commission early in 1916.

Population Studies and Sewer Gagings at Philadelphia, Pa.

Since 1907 studies have been under way looking to the preparation of a plan for the collection, treatment and final disposal of the sewage of the entire city of Philadelphia. In order to design intercepting sewers and sewage treatment works it was necessary to determine the present and estimate the future amount of sewage discharged from each sewer outlet. This was done by determining the present and estimating the future population upon each drainage area in the city and estimating the dry weather flow of sewage per capita per day to be used for each area. The method employed in making the population studies and the data obtained by the gaging of sewers are here given from information taken from the latest (1912) annual report of Mr. W. L. Stevenson, Assistant Engineer, of the Sewage Disposal Division of the Bureau of Surveys.

Population Studies.—The population of the entire city in 1950 was estimated by projecting forward the curve platted from the census returns from 1790 to 1910. This indicated a population of 3,000,000 in 1950. In 1851 Philadelphia was divided into 21 wards which, as the population increased, have been subdivided until there are now 47 wards in the city.

The census returns from the areas represented by the original 21 wards were next platted from 1860 to 1910 and each projected forward to 1950. These figures were then balanced so that their sum was 3,000,000, as determined previously for the population of the entire city in 1950.

Each of the 21 original wards was then subdivided into the present wards in a similar manner, but the balancing in this case was based somewhat upon the known characteristics of the development in each territory.

From the latest atlases the areas of undevelopable land—parks, cemeteries, etc.—were determined and a density per acre of developable land found and applied to the drainage area of each sewer by superimposing it upon the map showing the unit census enumeration districts. This gave a present and estimated population in 1950 contributing sewage to each sewer outlet.

Sewer Gagings.—To determine the amount of sewage per capita per day, gagings were made of the dry weather flow of ten characteristic sewers, the population contributing sewage above the gaging point being determined from the census of 1910.

These gagings were made over a period of 24 hours in order to show the fluctuations in the flow, and from them factors were obtained for the daily average, daily minimum and daily maximum flow of sewage per capita. The data obtained by the gagings are shown herewith in Figs. 1-6 and are summarized in Table I.

TABLE I.—SUMMARY OF DATA OBTAINED FROM GAGINGS OF DRY WEATHER SEWAGE FLOW, MADE IN 1910, PHILADELPHIA, PA.

			Area in Acres		Population Census 1910		Average Discharge per 24 hours. Gallons per Day			Cu. Ft. per Sec.		
Name of Area.	Character.	Point Gaged.	Total	Settled 1910	Total	Per Settled Acre	Total	Per Settled Acre	Per Capita	Total	Per Settled Acre	Per Capita
Thomas Run	Residential, mostly pairs of two and three-story houses	Pine St., W. of Conestoga	320	240	15,012	62.5	3,410,000	14,200	227	5.26	.0220	.000350
		Conestoga St., S. of Pine	426	337	21,677	64.0	3,320,000	9,860	153	5.14	.0153	.000237
		Outlet at Cobb's Creek	1,094	627	36,336	58.0	6,175,000	9,850	170	9.60	.0153	.000265
Pine St.	Residential, mostly solid four to six-story houses	Pine St., between 25th and 26th Sts.	160	156	15,152	97.0	4,105,000	26,300	271	6.35	.0407	.000419
Shunk Street	Residential, mostly rows of two and three-story houses	Shunk St., at Bancroft St.	208	208	25,754	123.0	2,190,000	10,500	85	3.39	.0163	.000132
		Shunk St., W. of 18th St.	331	331	37,916	114.0	3,516,000	10,600	93	5.44	.0164	.000143
Lombard St.	Residential, tenements and hotels	Lombard St., at 3rd St.	147	145	16,363	113.0	5,030,000	34,750	308	7.78	.0536	.000475
York St.	Residential and manufacturing	York St., at Cedar St.	358	354	33,340	94.0	12,754,000	36,000	383	19.70	.0556	.000591
Market Street	Commercial	South Side, W. of Strawberry St.	58	36*	The population contributing sewage is not shown by the census figures.		3,547,800	99,250	...	5.50	.1534	
		North Side, at Bank St.	123	80*			7,608,000	92,800	...	11.80	.1435	

*This area is practically entirely built up. The settled area is "total" minus street area.

The factors thus obtained from differently developed drainage areas were then applied to the estimated populations upon ungaged drainage areas having similar characteristics as to developments.

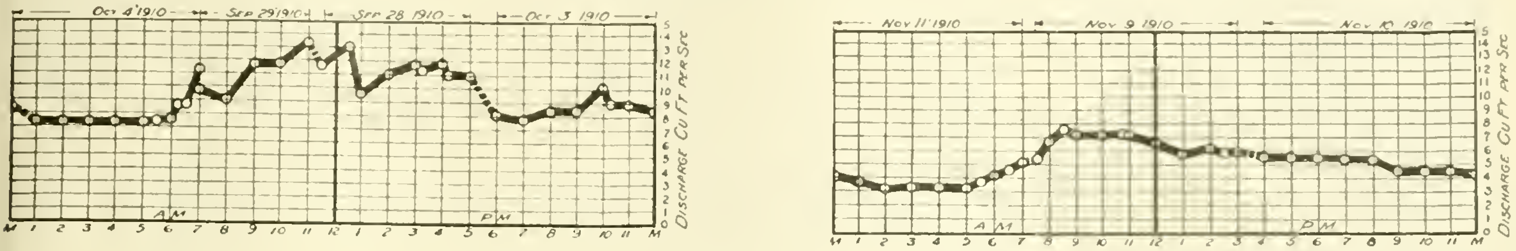
The coefficient of roughness of the sewers gaged, based upon dry weather flow, are shown in Table II.

TABLE II.—COEFFICIENT OF ROUGHNESS OF SEWERS GAGED DURING 1910—BASED UPON THE DRY WEATHER FLOW, PHILADELPHIA, PA.

Name of Area.	Location.	Construction.	Condition.	Year Built.	Coefficient of Roughness as Determined from Sewer Gagings.
Thomas Run	Pine St., W. of Conestoga St.	6 ft. 6 in. Circular Brick	Clean, good condition	1903	.016
Thomas Run	Conestoga St., S. of Pine St.	7 ft. 6 in. Circular Brick	Good condition	1902	.0115
Thomas Run	Outlet at Cobb's Creek	11 ft. Circular Brick Stone Block Bottom	Clean, good condition	1903	.017
Pine Street	Pine St., between 25th and 26th Streets	8 ft. Circular Brick	Good condition.	1830	.011
Shunk Street	Shunk St., at Bancroft St.	7 ft. Circular Brick	Dirty, fair condition	1893	.023
Shunk Street	Shunk St., W. of 18th St.	9 ft. Circular Brick	Clean, fair condition	1893	.019
Lombard Street	Lombard, at 3rd St.	6 ft. Circular Brick	Good condition	1883	.010
York Stteet	York, at Cedar St.	7 ft. Circular Brick	Good condition	1902	.012
Market Street	South Side, W. of Strawberry St.	7 ft. x 7 ft. Concrete Vit. Shale Brick Bottom	Clean, fair condition, bottom, uneven	1907	.0145
Market Street.	North Side at Bank St.	7 ft. x 7 ft. Concrete Vit. Shale Brick Bottom	Good condition	1907	.014

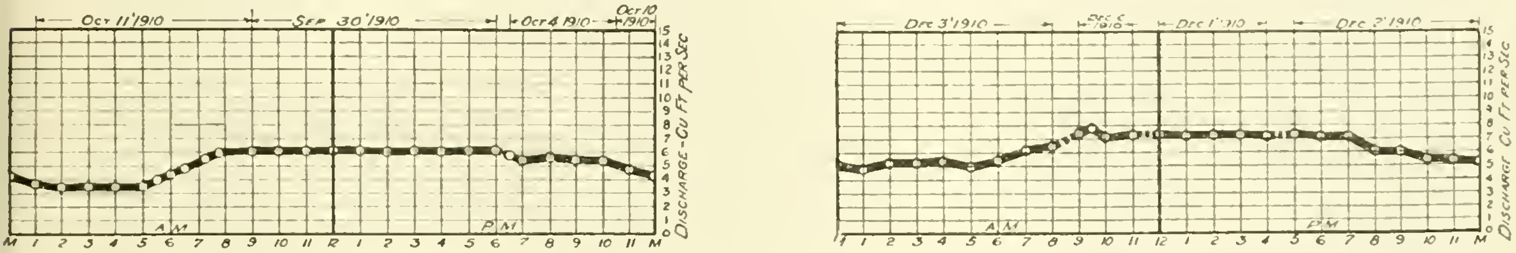
Boring Blast Holes With Diamond Drill.

The diamond drill is being used by the Hercules Sandstone Company at Tenino, Wash., for the drilling of deep blast holes in its quarry. This company has a contract to supply stone for a Government jetty at Gray's Harbor, and large sized stone is required for this work. The quarry face is about 70 ft. high. The method at first used consisted in drilling "coyote" holes horizontally at the base of the cliff. This resulted in shattering the rock badly and breaking much of it too small for use, thus causing a high operating cost. During the fall of 1913 the company installed a Sullivan class "C" diamond core drill, with "A" fittings, boring a hole 1 13/16 ins. in diameter and normally removing a 1⅛-in. core. With this drill holes are bored to an average depth of 63 ft., 30 ft. back from the face, and 9 ft. apart. After boring the hole, a special tool is run down, cutting a groove on each side, in line with the line of holes. These grooves aid in splitting the rock, which has a favorable grain, from one hole to the other. The holes are sprung with dynamite and then loaded with black powder to about 10 ft. from the top. When shot, the whole face is thrown over into the quarry and breaks into very large pieces. The average progress by the diamond drill has been about 53 ft. of hole per day, which indicates the favorable character of the stone. The cost of drilling and grooving together is given as 28 cts. per foot. The wear on the diamond is practically nothing.



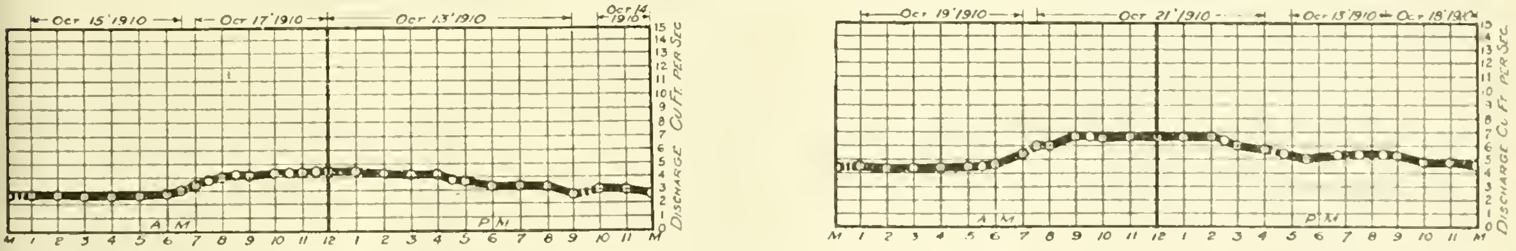
(a) Gaged at the outlet at Cobb's Creek. (b) Gaged on Conestoga St., 250 Ft. South of Pine St.

Fig. 1. Twenty-four House Gagings of Dry Weather Sewage Flow on Thomas Run Sewerage System, Philadelphia, Pa.



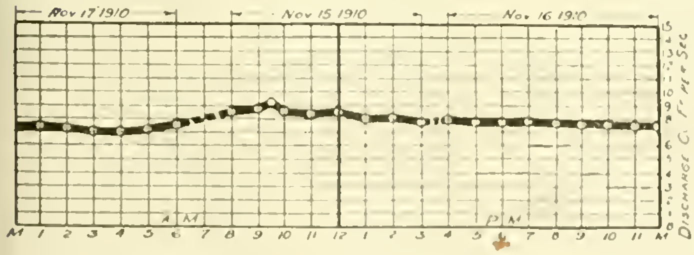
(a) Gaged on Pine St., 38 Ft. W. of Conestoga St. (b) Gaged on Pine St., between 25th and 26th Sts.

Fig. 2. Philadelphia Sewer Gagings—(a) Thomas Run System and (b) Pine Street Sewer.



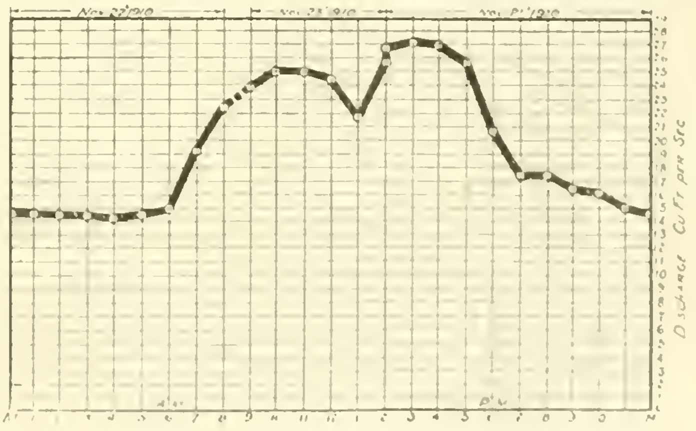
(a) Gaged on Shunk St. at Bancroft St. (b) Gaged on Shunk St., W. of 18th St.

Fig. 3. Philadelphia Sewer Gagings—Shunk Street System.



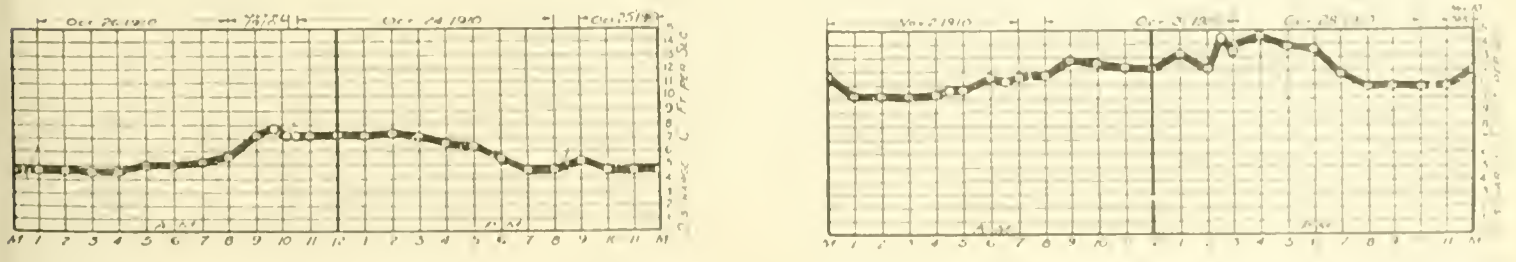
Gaged on Lombard St. at Third St.

Fig. 4. Philadelphia Sewer Gagings—Lombard Street Sewer.



Gaged on York St. at Cedar St.

Fig. 6. Philadelphia Sewer Gagings—York Street Sewer.



(a) Gaged on Market St., 52 Ft. W. of Strawberry St. (b) Gaged on Market St. at Bank St.

Fig. 5. Philadelphia Sewer Gagings—Market Street System.

General

The Determination of Unit Prices of Material for Purposes of Valuation of Plant.

Contributed by R. V. Achatz, Instructor in Telephone Engineering, Purdue University.

One of the first questions which must be decided in making a valuation of a physical plant is what unit costs of material and labor are to be used. In making a valuation on the cost of reproduction basis, it is agreed that these costs should be present costs but there is a question of just what is present cost. In the case of labor present cost is a cost based on the current wage scale, but in the case of material it is quite generally agreed that the market price on a given date, particularly for those materials which are subject to constant fluctuations in price, cannot always be used with justice. Many writers in discussing this question have said that, in case of materials subject to price fluctuations, the market price on a given date should not be used but an average price over a number of years past, usually five or ten, should be adopted. There immediately arises a question as to the propriety of using an average of past prices in a valuation on the cost of reproduction basis, and there is also a large question as to whether such an average actually represents a fair present price. It has been proposed that the present normal price can be determined by plotting the prices for a number of years past and drawing a smooth curve representing an average of the prices as shown by the yearly price curve.

In order to make a study of the different methods of determining unit costs of materials the prices on three metals, copper, tin and lead, were used. These metals were adopted because prices were available for many years past, because of their importance in telephone and other electrical properties, and because the prices are representative of three types of price variation. Copper prices have fluctuated continuously and sometimes violently with a general tendency toward increase. Tin prices have also fluctuated considerably and also have shown a marked increase in the past fifteen years. Lead prices have in general been stable and show little if any tendency toward change. Table I shows prices for Lake Ingot copper. The prices are taken from a leaflet called "Copper History" published by the Rome Wire Co., Rome, N. Y., and are based on actual prices paid. These prices are slightly higher than prices given in the "Iron Age" but the difference is usually only a small fraction of a cent per pound. Tables II and III are prices of tin and lead respectively and are compiled from data given in "Iron Age" in the first issue of the year for several years past

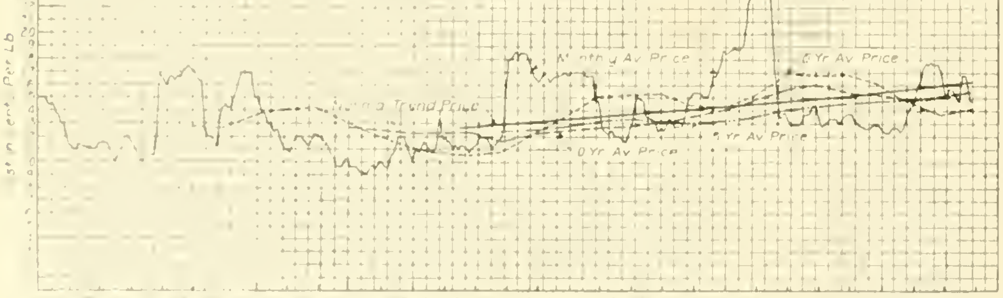


Fig. 1. Variation and Trend of Prices for Copper for Period 1884 to 1913, Inclusive.

In making the study curves were plotted for the monthly average prices of the metal (not given in the tables) by plotting at the abscissa representing each month, the average price for that month and connecting the points by straight lines. In a similar way curves were plotted for the average prices for 5, 10 and 15 year periods by plotting at the close of each year the average of the prices for the period preceding and, as before, connecting the points by straight lines. A smooth curve was also drawn representing a mean between the higher and lower changes in the monthly average prices. Theoretically there should be equal areas, above and below, between this curve and the monthly average curve but in practice it can be drawn by eye with sufficient accuracy. This curve has been designated a "normal trend price" curve.

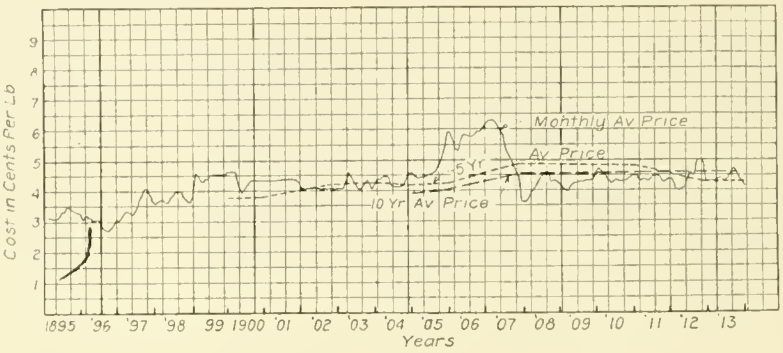


Fig. 2. Variation and Trend of Prices for Lead for Period 1895 to 1913, Inclusive.

In Fig. 1 are shown the curves of copper prices. These prices extend from 1884, the earliest period that prices were available, to the present. It will be noted that the prices have had peaks at periods of from seven to ten years apart, the highest peaks occurring in 1889, 1899, and 1907, followed by periods of comparatively low prices. The dotted curve represents successive five years averages and was plotted as described in the preceding paragraph. This curve also has peaks coming at the frequency of the peaks in the monthly average curve but displaced so that the maximum points in the five year average curve come later than those of the monthly average and occur during times of low market prices. For example, at the close of 1909 the market price of copper was 13.75 cts. per pound, while the five years average was 16.808 cts. per pound. At the close of 1912 the market price was 17.75 cts. per pound, while the five year average was 13.91 cts. The high average for the five years ending with 1909 was due to the influence of the very high peak in the market in 1906-7 and the low average for the five years ending in 1912 was due to the abnormally low prices following the 1907 peak, the influence of that peak on the five year average having passed. It is at once seen that a five year average price at either of these times would be unjust, the price in 1909 being too high to be just to the public and the price in 1912 too low to be just to the utility.

The ten year average prices of copper are shown in Fig. 1 by the broken line. This curve has peaks in much the same way as the five year average curve but the differences from time to time are not so great. The ten year average at the close of 1911 is 1.1 cts. less than at the close of 1908. The use of this average would be less open to objection than the five year average on account of smaller variation from time to time.

The 15 year average is shown on Fig 1 by the dot and dash line. It is entirely free from variation due to the influence of peaks in the market price and shows a general tendency upward. Its use might be objectionable on account of the influence of a period of low prices a long time ago.

The smooth curve which has been called the normal trend price has been drawn to represent an average of the market prices. At any given time the price read from this curve may be considered to be the price of copper under normal conditions. A buyer of copper in the future would expect to find prices following this curve more or less closely. The price as indicated by this curve was 15.2 cts. at the close of 1909, 15.8 cts. at close of 1911 and 16.3 cts. at close of 1913.

Figure 2 shows the variation in prices of tin. This metal like copper has been subject to considerable fluctuation in price and has also more than doubled in price in the past fifteen years. The curves representing the 5, 10 and 15 year averages and the trend curve have been drawn in the same manner as in case of copper. The average curves are free from the variation from time to time noted in the average curves for copper but it will be noted that, with the exception of a few months in 1904 and the years 1908, 1909 and part of 1910 for the five year curve and a few months in 1908 and 1909 for the ten year curve, the average curves are consistently lower than even the lowest points in the market prices. This brings out more clearly the objection mentioned above in the discussion of the prices of copper that average prices might be too greatly influenced by previous low prices to be used if the basis of the valuation is the cost of reproduction. If the basis is original cost of course this objection disappears. This discussion, however, has been predicated on the use of cost of reproduction. In the case of tin the normal trend curve at the close of 1913 shows a price of 45.9 cts. per pound while the five year average, the highest of the average curves, shows 39.468 cts. per pound. It is evident in this case that the use of any average price is unfair.

The variation in lead prices is shown in Fig. 3. The monthly average price curve shows that lead prices have been practically constant since 1899 with the exception of the abnormally high prices in 1906-7. This peak was caused by no particular condition of the lead market at that time but by the general inflation of commercial affairs during those years. The average price curves have been drawn as before. It will be noted that the 10 and 15 year curves still show that influence of the 1907 high prices while the five year average has passed beyond this influence and has come very near to the normal price. The normal trend curve has not been drawn to avoid confusion in the figure but if it were drawn it would be a straight line parallel to the X-axis at 4.4 cts. This is exactly the same as the five year average at the close of 1913. There have been periods in the past, however, when the use of the five year average would have been incorrect. For instance, at the close of 1909 this average was 4.926 cts. per pound while at the close of 1912 it was 4.379 cts., a decrease of about 11 per cent.

TABLE I.—AVERAGE PRICES OF "LAKE" INGOT COPPER.

(Prices compiled from pamphlet, "Copper History," published by Rome Wire Company, Rome, N. Y.)

Period ending Dec. 31—	Av. for the year, per lb.	Av. for 5 years, per lb.	Av. for 10 years, per lb.	Av. for 15 yrs., per lb.
1881	$0.14031	......	......	......
1885	.11166	......	......	......
1886	.111458	......	......	......
1887	.11323	......	......	......
1888	.16781	$0.12889	......	......
1889	.137395	.12831	......	......
1890	.15812	.13760	......	......
1891	.13093	.14150	......	......
1892	.11625	.14210	......	......
1893	.10781	.13010	$0.12950	......
1894	.095416	.12171	.12501	......
1895	.10812	.11171	.12465	......
1896	.10979	.10748	.12449	......
1897	.11333	.10691	.12450	......
1898	.12062	.10945	.11978	$0.12664
1899	.17802	.12598	.12384	.12533
1900	.16656	.13766	.12468	.12899
1901	.16729	.15116	.12832	.13271
1902	.12135	.15117	.12883	.13325
1903	.13791	.15423	.13184	.13126
1904	.13250	.14512	.13555	.13093
1905	.16093	.14399	.14083	.13122
1906	.19812	.15014	.14966	.13560
1907	.21177	.16825	.15951	.14197
1908	.13540	.16776	.16099	.14381
1909	.13420	.16808	.15660	.14640
1910	.13135	.16217	.15308	.14794
1911	.12750	.14802	.14910	.14912
1912	.16708	.13910	.15368	.15271
1913	.15833	.14369	.15572	.15522

From the study of the curves and the above discussion the following conclusions are drawn regarding the use of the different methods of determining unit costs of material for valuation purposes:

TABLE II.—AVERAGE PRICES OF TIN.

(Prices compiled from Iron Age, first issue of each year. Prices are on carload lots, New York market.)

Period ending Dec. 31—	Av. for the year, per lb.	Av. for 5 years, per lb.	Av. for 10 years, per lb.	Av. for 15 yrs., per lb.
1895	$0.14033	.....	......	.....
1896	.13325		.	..
1897	.13592	....	.	. .
1898	.15613	.... .	. .	. . .
1899	.21853	$0.16283	.	
1900	.29579	.19392		
1901	.26326	.21993		
1902	.26633	.24600		
1903	.27973	.27073	.. ...	
1904	.27658	.27631	$0.21959	
1905	.31438	.28005	.23699	
1906	.39666	.30764	.26333	
1907	.38239	.32995	.28798	
1908	.29438	.33288	.30180	. . .
1909	.29678	.33692	.30663	$0.25869
1910	.34239	.34252	.31129	.27217
1911	.42742	.34867	.32780	.29185
1912	.46350	.36489	.31742	.31362
1913	.44332	.39468	.36378	.33276

(1) The use of the market price of materials, especially those which are subject to price fluctuations is likely to be unfair.

(2) Average prices for a period of five years previous to the valuation are unreliable on account of the influence of periods of high market prices which tend to raise the five year average during the period of low prices that usually follows a period of high prices and the corresponding decrease in the five year average following a period of low prices even after prices have again increased.

(3) An average of the prices for a period of ten years previous to the valuation fluctuates in a less degree than the average for five years, but may be lower than a fair price or higher than a fair price due to the influence of a period of low or high prices many years before

(4) An average of the prices for 15 years may be unfair for the same reason as the second given under the ten year average, viz.: the effect of prices of many years previous.

(5) An average smooth curve can be drawn, taking into consideration market prices for a number of years past and also successive average prices for periods of 5, 10 and 15 years which will represent the normal present price of the material. In general the use of this curve as a basis of unit price of material will be more fair and less open to objection than any average price.

(6) Before unit prices are adopted it is necessary to make a study of the past and present prices of materials, particularly the more important ones which may represent a large portion of the total cost of material in the plant. Such materials would be poles, copper wire, lead covered cable, duct materials and Portland cement in the case of telephone, street railway and other electrical properties, and cast iron pipe in case of gas and water plants.

There may be some objection to a price based on the trend curve on account of the fact that the judgment of the appraiser is brought into its determination, but it must be remembered that a valuation is an estimate of cost to reproduce a given plant and the entire result is based on engineering judgment. Furthermore a result arrived at by the use of well trained judgment after considering all the facts is more likely to be fair than an average over an arbitrary period without further consideration.

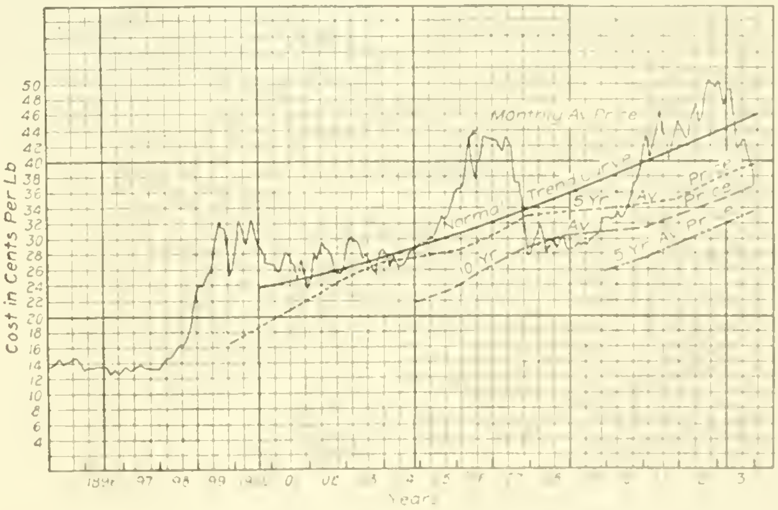


Fig. 3. Variation and Trend of Prices for Tin for Period 1895 to 1913, Inclusive.

The use of such a trend curve as a basis for the unit price of a material is not new although little has been written concerning it. As far as the writer's knowledge goes the most important appraisal that has been made public in which this method was used is the appraisal of the property of the Chicago Telephone Co. made by H. M. Byllesby and Co. and the Arnold Co. in 1911. It is true that Prof. Edward W. Bemis, who was conducting the investigation of rates for the Chicago City Council in this case mentions the price of 16 cts. used in the appraisal as a doubtful point but he accepted the valuation based on this price. Prof. Bemis' comment is as follows:

> Among the doubtful points in the appraisal may be mentioned the price of copper used by the appraisers. Copper was taken at 16 cts. a pound as of Aug. 1, 1911. The following table gives the average price for each of the 19 years, 1893-1911, inclusive, as taken from the Iron Age by the Chicago Telephone Company. These prices are a little lower than those given in the Telephone Directory of the telephone industry, 1912 edition (page 333):
>
> (Table of copper prices omitted.)
>
> The above table would indicate that 16 cts. was high. The company has, however, established an elaborate trend curve of prices of copper to show that 16 cts. a pound is in line with the trend or tendency of prices, at the time of the appraisal. Since the earliest quotations on hand, beginning in 1883, copper has averaged less than 16 cents every year save in 1888, 1899-1901, inclusive, and 1905-1907, inclusive. During those years only 309,433 miles of wire out of 807,571 in use at the close of 1911, or 38 per cent, were laid. It would be easy to show that copper had not from the beginning averaged over 15 cts. per pound, and had not even averaged that for any period of 5 or more years before the appraisal. At the same time, if the test of value is not to be the actual cost or recent costs, but probable costs of material and labor during the next five years, then 16 cts. may be accepted as a probable price.

TABLE III.—AVERAGE PRICES OF LEAD.

(Prices compiled from Iron Age, first issue of each year. Prices are on carload lots, New York market.)

Period ending Dec. 31—	Av. for the year, per lb.	Av. for 5 years, per lb.	Av. for 10 years, per lb.	Av. for 15 yrs., per lb.
1895	$0.03235	......	......	......
1896	.02971	......	......	......
1897	.03566	......	......	......
1898	.03769	......	......	......
1899	.04469	$0.03802	......	......
1900	.04390	.03833	......	......
1901	.04355	.04110	......	......
1902	.04093	.04215	......	......
1903	.04266	.04315	......	......
1904	.04360	.04293	$0.03947	......
1905	.04898	.04394	.04114	......
1906	.05800	.04683	.04397	......
1907	.05408	.04946	.04581	......
1908	.04224	.04938	.04626	......
1909	.04300	.04926	.04609	$0.04274
1910	.04461	.04839	.04617	.04349
1911	.04436	.04566	.04625	.04453
1912	.04476	.04379	.04663	.04512
1913	.04401	.04415	.04676	.04556

In advocating the use of the trend curve as a basis for determining the unit prices of material for purposes of valuation, it can not be said that this method will give, in every case, results which are absolutely fair and uniform. Judgment is required in its use and unforseen developments may change the trend of the price of any material but it is believed that the use of this method will give unit costs that are more fair to all parties concerned than any other method that has been used.

Heat Insulation Tests.—Tests are being conducted by the University of Illinois for the purpose of bettering the heat insulating qualities of the walls of railway mail cars. These tests are being made for the New York Central Lines, and are under the direct supervision of Professor Arthur C. Willard, who is in charge of the heating and ventilating courses at the university.

Transmission Tower Line Location—Modern Field and Office Practice.

Contributed by A. B. Cudebec, 503 Kearns Building, Salt Lake City, Utah.

A period of 15 years will cover almost the entire development of modern transmission at pressures above 11,000 volts. Possibly the first successful transmission system in this country at 40,000 volts was put into service by the Telluride Power Co., which began operations in Colorado. From this small beginning the art has now developed until many water powers formerly valueless, except for turning the wheels of local mills, due to their isolated position, are now harnessed to mighty generators, the energy from which is carried over the modern transmission lines to congested centers of population.

The last word in long distance transmission doubtless finds expression in the Big Creek California line, which transmits its power a distance of 240 miles with a line pressure of 150,000 volts. As an evidence of the sturdy character of modern construction methods, it is reported that during the recent floods in California, when whole railroad systems were tied up, this power system maintained uninterrupted service.

Executives are but slowly learning that transmission line location needs experienced men, in order to avoid the resulting operating troubles oftentimes caused by the blunders of cheap inexperienced locators. A locating engineer equipped with judgment and tact may easily save thousands of dollars to the right of way men, simply by courteous, thoughtful treatment of the property owners along the line.

The generally accepted idea that transmission line location is of little consequence is reflected in the very few books treating upon transmission construction. In one of the most recent books a few pages only are devoted to this item. A statement in one recent publication which has received favorable comment from critics reads substantially as follows: "A surveyor with a stadia rodman or two will usually fill the needs of a locating party." This statement is pitifully inadequate. One example of this procedure will suffice:

A great corporation recently started the construction of a tower line, using a local railroad surveyor for locator, with the result that a crew of graders had to follow the wire stringing crew to grade off the high points between towers in order to secure safe clearance. At one point 5,000 cu. yds. of earth were moved. This surveyor lived in the construction camp and laid out the line and selected tower locations only a few miles in advance of the foundation crews. The line cost above $3,000,000, and its general location and tower sites should have received months of study by trained men.

We are referring only to high tension of 60,000 volts and upwards, and it might be well to state that the lower tension lines so common to urban and suburban districts present few, if any, of the problems of high tension location.

The principal forces which influence the location might be classified briefly: (1) Type of supporting structure and limiting span lengths, (2) cost of right of way, (3) cities or villages which must be approached or avoided, (4) proximity to existing telephone or railroad signal circuits, (5) character of foundations which may be encountered.

Before an important line is located, the exact type of design should be settled by the designers. The limiting sizes of all angles and the approximate cost of inserting an angle should be known.

Oftentimes a line is surveyed before one or both of its terminals are selected. This usually leads to costly line revision and can usually be avoided if the point is put before the executives with sufficient clearness.

Before the field parties are organized the engineer should invariably search for all available maps. The U. S Geological Survey topographic maps are of highest value, and those of local surveyors and county maps being of less use; a preliminary stadia line will usually pay well unless the location is in the western undeveloped districts or a U. S. Geological Survey map is obtainable. This stadia party can cover four or five miles per day and tie in all limiting features on either side of the line for a distance of one-half mile and can intersect angles upon important objects at much greater distances. This preliminary party should particularly observe points which might prove suitable for railroad crossings. A crossing angle greater than 60° should be selected and never located at points where the tracks are laid upon a fill. Most railroad systems insist upon rigid crossing specifications and they can usually enforce compliance with their wishes. Therefore, it is usually wise to select a crossing with spans not greater than 200 ft. and a rail clearance of 35 ft.

Many of the systems which are at present operating with one line contemplate the construction of a duplicate line as soon as earnings will warrant the expense. It would appear to the writer as being good business to locate the second line over an entirely new right of way, separated far enough to minimize the chances of both lines being put out of commission by unusual storms. The chief objections to this plan are the increased right of way cost and additional cost of line patrol.

In any event, the best location for a second line should be selected by reconnoissance and all facts in connection with the duplication of lines should be laid before the executives in ample time to allow a definite plan to be evolved. Changes in general plans after the work is well started often incur serious mixups and the resulting costs of construction may be higher.

Following the completing of the preliminary line and the projection of the location, the locating party is ready to start work. The organization is similar to railroad location work—a transit and level party, which can be split up into two land survey parties at regular intervals.

Few, if any, of the early pole lines have well described rights of way. The surveyor was usually accompanied by a right of way agent who paid the land owners a stipulated price per pole, took a receipt for the money and the transaction was closed. This procedure does not fulfill the requirements of the modern corporation lawyer, who insists upon good and sufficient legal titles to all rights of way, etc. With the rapid increase in the number who invest in the securities of public utilities, together with the constant tendency toward publicity and regulation, it behooves the management to be very sure of the legality of their titles to property. Regular right of way plats are now prepared which become a part of the permanent easements which are secured. These easements are filed in the county records similar to any property transfer by deed.

The locating party should collect definite data upon the following items: (1) Rock outcrops. (2) Limits of all swamps. (3) Extent and character of all timber crossed. (4) Details as to telephone, signal or telegraph circuits which are crossed or approached, including height of poles, number of cross arms, wires, etc. (5) A detail profile across all railroads and important highways, with angle of intersection and ties to nearest railroad mile post. (6) Accurate information as to slide slopes. The lack of these data often causes low ground clearance on the upper side if not properly allowed for in advance. (7) No dwelling house should be approached within 200 ft. and barns and outbuildings should be kept at least 150 ft. away. (8) Angle points should be selected so as to be suitable for tower locations, preferably upon a high point or upon a level and never in a hollow. (9) When two angles are necessarily close together, the distance between should be made such that the standard tower spacing may be maintained.

If the field notes are taken in loose leaf note books, the data can be mailed to the field office at the close of each day's work. Property plats and plans and profiles can be kept up within a few miles of the actual survey parties. The old-fashioned method of plotting profiles upon a continuous roll of profile paper is now being superseded by plotting the plan and profile upon sheets in two-mile sections. The plan is given below and the profile above. The profile should be unbroken across each separate sheet, except in special cases.

The selection of tower locations cannot be made by inspection but must be carefully studied, and is capable of endless combinations. A celluloid template is made, using the same scale as the profile paper used. Two curves are usually platted upon this sheet of celluloid, one curve showing the sag which the particular conductor selected will take, say, 100° F. and the second curve showing the sag assumed by the conductor at minimum temperatures—say 20° below zero. These are known as the "hot" and "cold" curves for this conductor.

By means of this template the maximum spacing and economic location of all towers may be determined. The details as to the use of this curve are described in the "Electrical World," Sept. 11, 1911, by Mr. J. M. Viehe. As soon as the tower locations have been determined in the office, a complete tabulation of tower data should be made and handed to the construction superintendent. This data should include tower number, survey station number, type of tower, length of span to next tower, angle turned, if any, type of footing, owner of property and any special information such as railroad crossing, special protections, etc. A bound folder containing blue prints of all of the plan and profile sheets, together with a general key map, should be supplied to the construction superintendent. With this information in hand the construction superintendent is in a position to make out a complete shipping schedule. The final staking out of tower locations needs the personal supervision of the locating engineer, as numerous local changes may be necessary to save expensive excavation. A tower might fall upon a pile of boulders, which could be avoided by moving 20 ft., and excessive grading for drainage or good footings might be reduced by inserting a side hill extension to the tower legs. If a systematic routine of a notification to all concerned is followed by the engineer making the changes, much delay and friction with construction forces may be avoided.

Steel tower lines cost from $6,000 to $14,000 per mile, according to the weight of the conductors and steel supports, while the cost of surveys and location will amount to $50 to $100 per mile. Does it not seem good policy to engage experienced engineers on the basis of cost comparison alone, neglecting all other arguments?

The general tendency of transmission practice today is, however, toward permanent, long-lived construction, but it is doubtful whether this is warranted at this stage in the development of the art. If the next 20 years should show as great advances as have been made during the past two decades, the great tower lines of the country will be obsolete before one-third of their lifetime has elapsed.

A Gasoline Locomotive for Contractors and Light Industrial Service.

(Contributed.)

Baldwin gasoline locomotives, built in accordance with patents granted to A. H. Ehle, have now been in service for several years, and have fully demonstrated their efficiency in light industrial and contractors' service. The success of these machines is, unquestionably, largely due to the fact that they follow steam locomotive design where practicable, and embody features which have proved successful in this class of power.

The illustration shows a Baldwin gasoline locomotive recently built for the Georgia Coast & Piedmont R. R. This engine is used in light switching and road service, and is capable of handling a load of 30 tons over grades of 0.5 per cent and curves of 300 ft radius. The track gage is standard, and the locomotive has a nominal speed of six miles per hour on low gear and twelve miles per hour on high gear. The hauling capacity given above applies to the latter speed.

This locomotive is propelled by a four-cylinder, four-cycle water cooled engine, of 50 H.P. nominal capacity. The power developed by the engine is transmitted to the wheels through a system of shafting, gearing and side rods. This furnishes a positive drive, with large journals and bearings throughout; and all the parts are of ample strength for severe service. The engine and transmission are covered by a hood, on top of which the gasoline tank is placed. This tank has a capacity of 30 gals., an amount sufficient to propel the locomotive a distance of 50 or 60 miles, under average working conditions.

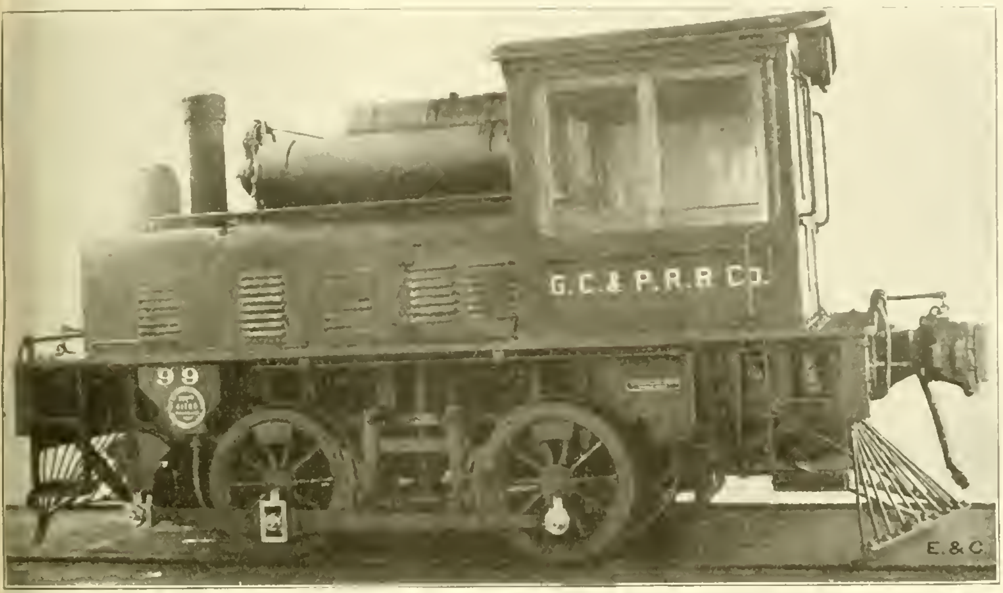


Fig. 1. Gasoline Locomotive for Contractor's Service.

The frames are steel castings, of the familiar bar type. The open construction of these frames facilitates the ventilation of the engine and the inspection of the transmission. This design of frame is very strong, and is also convenient for the suspension of the brake work, sand-boxes and other fittings.

This locomotive is required to handle truck line rolling stock, and its equipment has received special attention. Electric headlights, iron pilots, and M. C. B. automatic couplers are provided at each end. Air-brake equipment is applied, air being furnished by a compressor driven from the main engine. The air reservoirs are mounted over the hood. Another important feature is a self-starter for the main engine. This is driven by an electric motor, which receives its current from a storage battery. The latter is automatically charged by a generator driven from the main engine. The battery also furnishes current for the head-lights and cab-lights. The equipment also includes a hand brake, a bell, an air-operated whistle, and four sand-boxes with spouts to all the wheels.

Special attention has been given to the arrangement of the cab and cab-fittings. The cab is roomy, with large side windows, and the fittings are so arranged that the operator can handle the locomotive without leaving his seat. Steps and grab irons are arranged, as far as possible, in accordance with the requirements of the Interstate Commerce Commission.

The appearance of the locomotive is strongly suggestive of steam practice. The effect is more pronounced because of the position of the muffler, which stands vertically and looks like a smokestack. This device is most efficient, rendering the locomotive almost noiseless in operation.

These machines possess special advantages for light switching and industrial service. They are self-contained like a steam locomotive; are reliable and simple in operation, and can be easily handled by one man. An advantage which is most apparent in intermittent service is that they consume no power while standing idle. Here the self-starter is particularly effective, as when the locomotive is idle for only short periods of time, the engine can be stopped and started without labor on the part of the operator.

Transporting Pipe Line Material Across a Desert by Motor Truck.

To the Editor: Your attention is called to an article in your issue of Feb. 4, 1914, about transporting pipe line across the desert with a motor truck, which is only about half the story, and would be in its present state entirely deceptive as to the actual facts.

You have stated the steep grades and altitude as being 8,500 ft. above sea level, while this line goes through the Tejon Pass, which is slightly over 4,000 ft.—about 4,200 ft.

These trucks were operated under a subcontract from Mahoney Bros., the contractors, for about three weeks, when it was found that the freight piled up so fast that it was necessary to abandon them and put on about 680 mules. These did the work satisfactorily, which is rather strange on account of your statement tending to depreciate their service because of the great heat, soft roads, etc. Before the auto trucks were laid off, they were stuck all along the route.

You also speak, with some embellishments, of the auto trucks going down the river bed. As a matter of fact, only one went down there, the autos being afterwards stopped at the top of the grade and the pipe hauled in with mules.

The conclusion arrived at, which is unfortunately omitted in your article, is that auto trucks are only good so long as the roads are in fair shape; otherwise they are worthless in comparison with mule teams. In this case the autos in one or two trips cut up the road so that it was impassable, and they spread out sideways on new roads with increasing difficulties, until it was necessary to abandon their service.

It would therefore seem that your article rather slanders the faithful, patient and efficient mule unnecessarily, when it should be praised instead of the motor truck.

Very truly yours,

Boyd L[illegible]

Victoria, B. C.
April 30, 1914.

Operation of Canal Terminal Locks.—A commission consisting of Mr. Carroll S. Smith and Mr. J. M. Barrett appointed by Mr. E. A. Drake, First Vice-President of the Panama Railroad Co., arrived at Panama on April 8, accompanied by a statistician, to make a study and report on the most practicable and effective plan for handling cargo to and from vessels at the Canal terminals.

A Note on Cost of Subaqueous Shield Driven Tunnels.

Leaving aside all special work and considering only the standard tunnel driven under compressed air, including excavation, cast iron lining, bolts, grouting, rust or cement jointing, concreting, the writer has endeavored, in so far as it is possible, to figure the cost of such a structure, reduced to unit cost per cubic yard of excavation, or, more exactly, per cubic yard of tunnel space. This unit cost represents, as nearly as can be ascertained, the cost to the contractor or constructing company, exclusive of cost of financing, of amortization, of interest on capital, and exclusive of profit. Neither does it include permanent way or equipment. This unit cost oscillates between $27.00 and $32.00 per cubic yard of tunnel. Among others, two examples were chosen, one being the Pennsylvania Hudson tunnels, 23 ft. in diameter, in which this unit cost was $31.00, and the other, the Paris Concorde tunnel, 25 ft. 6½ ins. diameter, the unite cost of which was $29.12. It may be noted, however, that there was very heavy concreting in the Pennsylvania tubes and very little in the Paris tunnel. In the Pennsylvania tubes there were also two special machined steel segments, spaced every 15 ft., which evidently increased the cost to a certain extent. The cost of labor was greater in New York than in Paris; on the other hand, the price of cast iron was double in France that in the United States. Taking the unit cost of the Paris tube at $29.12 per cubic yard, the various items making up that price are as follows:

Superintendence and labor	$10.00	34.34%
Plant charges	3.65	12.53%
Lining	11.00	37.77%
Bolts	0.78	2.68%
Blasting	0.4	0.13%
Grouting	0.83	2.89%
Jointing	0.11	0.37%
Wooden furring strips	0.05	0.17%
Concreting	0.70	2.40%
Cement facing	0.16	0.54%
Power	1.80	6.18%
Total	$29.12	100.00%

These costs are taken from a paper by P. A. N. Seurot before the Canadian Society of Civil Engineers.

Automatic Piston Rod Wiper for Rock Drills.

(Staff Article.)

One of the main causes of wear in the heads and cylinders and pistons of rock drills is the fact that, in spite of careful packing and close fitting of the working parts, some dust and grit is found to be carried into the head bushing or lining, and beyond the packing into the drill cylinder. When it arrives at this point, it roughs up or scores the cylinder wall, and even the surface of the piston and the piston ring. This is particu-

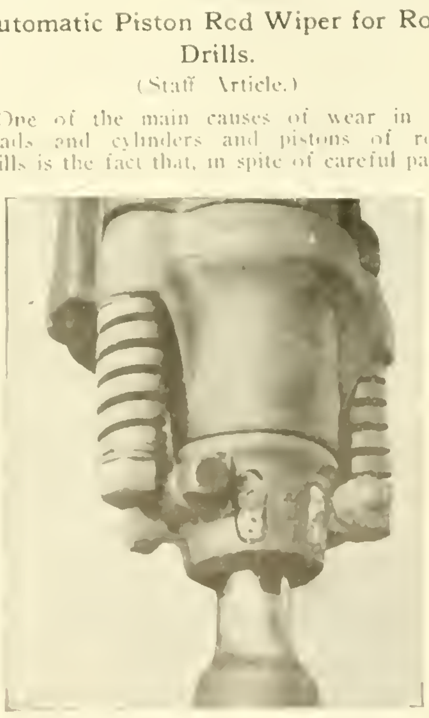


Fig. 1. Piston Rod Wiper for Rock Drills.

larly true in holes drilled above the horizontal, from which the dust and cuttings fall down upon the front head of the machine.

To reduce this wear a piston rod wiper, "illustrated" by Fig. 1, has been devised. As shown, this consists simply of a cage or frame, made in halves to bolt over the front head and around the piston of a rock drill, the purchase on the head being obtained by a groove cast in the halves of the taper lining, or cut in these parts, when the drill is in the field. This cage is kept packed full of waste that has been soaked in oil. As will readily be seen, this waste has the effect of keeping the piston rod, as it reciprocates, constantly wiped clean of any sludge, dust or moisture that may fall upon it during operation. The waste can of course be readily removed, and fresh waste inserted or additional oil put upon it when necessary.

We are indebted to the Sullivan Machinery Co., Chicago, Ill., for information of this new device.

Water Works

Cost of Obtaining Water from Wells at Owensboro, Ky.

A report on an investigation of the municipal waterworks at Owensboro, Ky., was made by Burgess & Long, consulting engineers of Columbus, O., on March 2, 1914. The investigation included studies not only of the physical condition of the plant but also of other important features, including the quantity of water required to supply the needs of the city; the design and general arrangement of the equipment provided to develop the water supply and to deliver it to the consumers; the usual methods of operating the plant; the adequacy and quality of the sources of water supply available; the general arrangement and adequacy of the distribution system; the cost of furnishing water required for domestic and industrial uses, and also for fire protection purposes; and the revenue derived from the water service rendered. The following matter is taken from the report and relates to water consumption, the well system, a general description of the waterworks plant, the operation of the waterworks plant, the amount and cost of power required to obtain water from the well systems, the quality of water obtained from the wells, and the cost of chemicals used in softening the well water supply:

WATER CONSUMPTION.

A study of the past records of operation at Owensboro indicates that no reliable data are at hand to show what the actual past water consumption has been. Probably the most accurate data available are the counter readings on the high service pumps. These counters are read generally several times a day but, so far as could be learned, no attempt is made to use these records for the purpose of computing or estimating the average daily consumption of water.

A feature of operation which tends to make such computations inaccurate is the fact that a very considerable quantity of water is used in the cooling jackets of the air compressors. This cooling water is taken from the high service force main, flows through the compressors, and back into the storage reservoirs. It appears in the quantity of water pumped, but, of course, is not delivered to the consumers. During the recent investigation the quantity of water so used for cooling purposes was estimated as correctly as the means at hand would allow and was found to be about 250,000 gals. daily.

The daily consumption, including water used to wash the filters in the softener, was found to be 1,424,000 gals. The present average daily consumption of water at Owensboro consequently is estimated to be about 1,400,000 gals. The census records of population indicate a considerable growth of the city, so that demands for the near future require that there be available at least 3,000,000 gals. of water daily in order to meet the demands of the service.

THE SOURCES OF SUPPLY.

When the waterworks were first constructed, in 1904-06, it was learned that at least two veins of water were available in the ground as possible sources of supply. The one was in a gravel stratum about 20 ft. thick and from 80 to 100 ft. below the surface of the ground; the other was in the rock strata underlying the gravel deposits. When the construction of the plant was begun, in 1904, it was thought that the upper gravel vein was subject to local pollution and was objectionable on this account. Moreover, the upper vein of water was thought to be much harder than that contained in the rock. Consequently the water supply was obtained from drilled wells driven into the rock to a depth of about 300 ft. below the surface of the ground. Water was pumped from the wells by air lifts operated under about 100 lbs. pressure. The cost of obtaining water from this depth was excessive and in 1907 the air lifts were raised to operate under from 50 to 60 lbs. pressure with a very much less consumption of fuel. About 20 such wells have been driven to date and practically all the land now owned by the city between the pumping station and the river is utilized for water supply purposes.

During recent years there has been an increasing demand for water, and in 1911-12 a system of four drilled wells was installed immediately adjacent to the pumping station to secure water from the upper or gravel vein of water. These wells are 8 ins. in diameter, about 101 ft. deep, and contain Cook strainers, each 20 ft. long. They are equipped with deep-well plunger pumps, driven by a 50-HP. electric motor, connected by shafting and gears to the well pumps. When sufficient water cannot be obtained from the deeper wells by the air lifts it is the present practice to augment the supply with water obtained from the new motor-driven well pumps.

An important question considered during the recent investigation was whether future additions to the water supply at Owensboro should be obtained from the lower vein of water in the rock or from the upper vein of water in the gravel strata. This question is largely one of the relative cost of operation and maintenance of the two well systems. In order to answer this question intelligently it was necessary to obtain data relative to the cost of power required to operate the two well systems; also to determine the relative cost of treating the two waters in order to obtain a supply of satisfactory quality.

Before discussing the results obtained from the tests of the two well systems, it is thought desirable to give a brief description of the equipment provided to obtain the water supply, to purify it and to deliver it to the city.

GENERAL DESCRIPTION OF WATERWORKS PLANT.

In addition to the two systems of wells and piping which have already been briefly described, the waterworks plant comprises the following equipment: Two air compressors furnished by the Rand Drill Co. and provided to lift the water from the deep wells; a water softening plant of 3,000,000 gals. daily capacity provided to purify the water as it is discharged from the wells into the receiving reservoirs; two covered circular storage reservoirs, each holding 1,000,000 gals.; two high service pumps each of 3,000,000 gals. daily capacity, built by the Platt Iron Works Co. of Dayton, Ohio, and provided to lift the water from the reservoirs and force it into the distribution system; three 200-HP. Atlas water tube boilers; and the necessary auxiliary equipment, including boiler feed pumps, vacuum pumps, chimney and other miscellaneous equipment.

Softening Plant.—The softening plant comprises devices for applying chemicals in solution; reaction or softening tanks where the water and chemical solutions are thoroughly mixed and retained for a period of one hour; two settling tanks provided to clarify the water which is retained in the tanks for a period of five hours; and wood-fiber filters provided to further clarify the settled water and to remove after-deposits.

Reservoirs.—The storage reservoirs, into which the softened water is discharged from the settling compartments of the softener, are two in number, 100 ft. in diameter by 17 ft. deep, each holding approximately 1,000,000 gals. For some reason the receiving reservoirs are built upon high ground between the pump station and the river. Consequently it is necessary to lift the water with the air lift or deep well pumps some 20 ft. higher than the high service pumps.

High Service Pumps.—The two high service pumps are duplicate machines, of the following principal dimensions:

Nominal capacity in 24 hours, 3,000,000 gals. (slippage 3 per cent.)
Capacity per revolution, 40 gals.
Maximum piston speed required, 225 ft. per minute.
Diameter of water plungers, 10½ ins.
Length of stroke, 24 ins.
Diameter of high pressure steam cylinder, 14 ins.
Diameter of low pressure cylinder, 24 ins.

The engines are of the cross-compound, crank and fly-wheel condensing type. One of the most important features of their design is the fact that a piston speed of 225 ft. per minute is required to develop the full capacity of the pumps. The duty guaranteed was 110,000,000 ft.-lbs. per 1,000 lbs. steam when delivering 2,100 gals. per minute against 120 lbs. pressure, with steam at 125 lbs. pressure. The duty to be 80,000,000 ft.-lbs. when operating at one-half capacity under the same conditions. The cost of the pumps was $18,320.

Air Compressors.—The two air compressors provided to operate the air lifts are of different design, although both were furnished by the Rand-Drill Company. Each compressor is of the cross-compound, crank and fly-wheel type.

The following are the principal items of interest concerning the two machines:

	(1)	(2)
Date of Contract.	Feb. 10, '05.	June 24, '05.
Type.	Rand-Corliss.	Rand Imperial Corliss.
Capacity, cu. ft.	2,300	2,300
Pressure of air, lbs.	100	100
H. P. Steam Cylinder, ins.	18	18
L. P. Steam Cylinder, ins.	34	30
L. P. Air Cylinder, ins.	29	28
H. P. Air Cylinder, ins.	19	17
Stroke, ins.	36	24
Revolutions per minute	91	135
Piston speed, ft. per minute.	456	540
Cost	$12,482	$9,924
Weight, lbs.	120,000	77,000

Guaranteed steam consumption of No. 1 = 15 lbs. per HP.-hour with steam at 125 lbs.

Mechanical efficiency of air compressor to be not less than 90 per cent.

Volumetric efficiency of air cylinder not less than 95 per cent.

Guaranteed steam consumption of No. 2 = 16 lbs. per HP.-hour.

Mechanical efficiency of air cylinder not less than 90 per cent.

Volumetric efficiency of air cylinder not less than 95 per cent.

Horse power developed by each machine = 400.

Starting pressure of No. 1 machine 125 lbs., of No. 2. 135 lbs.

Boilers.—The boiler house contains 3 200-HP. Atlas water tube boilers. Each boiler contains 87 4-in. tubes, 17 ft. 9 ins. long. The grates are 6 ft. 6 ins. by 6 ft. 1 in. Owing to the generally leaky condition of the boiler settings and, as stated, to the comparatively small capacity of the chimney provided, it is customary to operate the boilers with forced draught induced by an electrically-driven blower.

OPERATION OF WATERWORKS PLANT.

When the pumping station is in operation the general method of procedure is as follows

Water is pumped from the deep wells by the air lift, using compressed air delivered from the air compressor or is pumped from the shallow wells, using the motor-driven pumps, and is discharged into the raw water weir box of the water softener. From this point the water flows by gravity through the softener, receiving the chemical solutions on the way, and is discharged into the receiving reservoirs after having been softened and purified in the softener. From the receiving reservoirs the water is taken by the high service pumps and is discharged into the distribution system.

The following are the principal operating conditions of the plant: The air pressure maintained at the compressor varies from 47 to 60 lbs. per square inch, depending upon the height of the ground water at the wells, which in turn depends upon the stage of the river. The water pressure maintained in the distribution system by the pumps varies from 50 to 60 lbs. per square inch for the usual domestic service, and is increased to 100 lbs. per square inch for fire protection purposes. The steam pressure carried on the boilers varies from 130 to 135 lbs. per square inch. The vacuum on the discharge of the air compressors is about 24 ins. The large air compressor is operated at its nominal speed of from 85 to 90 revolutions per minute. The average speed of the pumps, when on domestic service only, is about 25 r. p. m. The maximum speed of the pumps, when furnishing 3,000,000 gals. daily, is 52.5 r. p. m., and the maximum pressure for which the pumps were designed is 120 lbs. per square inch. It should be noted that the air pressure generally carried is only one-half of that for which the air compressors were designed; also that the present speed of the pumps is about one-half of their maximum speed; also that the domestic pressure carried is about one-half of the maximum pump pressure. These features, of course, materially affect the efficiency obtained by the operation of the equipment, because the maximum efficiency is obtained at the nominal or maximum speed for which all of the equipment is designed.

Under the usual operating conditions the air compressors develop about 200 HP. and the high service pumps develop about 60 HP. In other words, about 3.3 times as much fuel is required to lift the water out of the wells as is required to pump it into the mains.

POWER REQUIRED TO OBTAIN WATER FROM WELL SYSTEMS.

The work done in raising the water from the wells by air lifts is equivalent to forcing 1,030,000 gals. daily against a head of 50 lbs. This requires 20.6 HP. theoretically. The air compressors, when operating at 90 r. p. m. against an air pressure of 100 lbs., develop 400 HP., and against 50 lbs. air pressure would develop at least 200 HP. The efficiency of the air lifts is, therefore, but 10 per cent.

The work done by raising the water from the walls by the motor-driven pumps is equivalent to forcing 920,000 gals. daily against a head of 100 ft. This requires 16.0 HP. theoretically. No devices are provided to measure the power delivered to the motor which drives the well pumps. This motor is said to be a 50-HP. motor and to operate at nearly full capacity. Assuming that 50 HP. are required to drive the pumps, the efficiency of the apparatus is 32 per cent.

COST OF POWER REQUIRED TO OBTAIN WATER FROM WELL SYSTEMS

During the recent investigation, on Jan. 10, 1914, measurements were taken of the steam consumed by the large air compressor. The method used was to weigh the condensate in barrels. Such measurements indicated that the steam consumed by the engine was at the rate of 6,000 lbs. per hour. The air pressure maintained was 48 lbs., vacuum 23.8 ins., steam pressure 133 lbs., speed of engine 88 r. p. m. No measurements were made to determine the amount of coal required to develop the steam.

Records have subsequently been kept to determine the daily coal consumption as indicated by the time dequired to consume a known weight of coal. The records indicate that a carload of 42.75 tons was consumed in 72 hours, and that during that time the compressor was shut down for 15 hours. The coal consumed by the compressor and pump is estimated at 40.30 tons in 57 hours and by the pump alone 2.45 tons in 15 hours. These figures are based on the assumption that the compressor required 3.3 times as much coal as the high service pump.

The fuel required to drive the compressor and auxiliaries is estimated at 0.54 ton per hour and to drive the pump and auxiliaries at 0.16 ton per hour. Coal costs $1.10 per ton delivered at the plant. On this basis, the cost of fuel required to operate the compressor per day is estimated at $14.25 and to operate the pump per day at $4.25. The cost of fuel required to pump 1,000,000 gals. from the deep wells is estimated at $13.70 and to pump 1,000,000 gals. into the mains at $4.20.

No data are at hand to show the cost of operating the motor-driven well pumps, except the fact that a 50-HP. motor seems to be required to do this work. The theoretical electric power required to pump 1,000 gals. against a head of 100 ft. is 0.312 kilowatt and to pump 920,000 gals. is 277 kilowatts. If the efficiency of the apparatus is but 32 per cent, the power required is 866 kilowatts. This costs about 1 cent per kw., so that the cost of power required for pumping 920,000 gals. is about $8.66 and the cost of pumping 1,000,000 gals. is about $9.42.

If the pumps were operated at a slower speed, it is the engineer's belief that an efficiency of 40 per cent should be obtained with the equipment provided, and under this condition the cost of power required to pump 1,000,000 gals. would be $6.93.

As shown above, the cost of power required to pump water from the air-driven wells is about double that required to pump water from the motor-driven wells, and over three times that required to pump it into the distribution system.

QUALITY OF WATER.

It is of interest to note that there is no very great difference in the quality of the water obtained from the two systems of wells. That pumped from the shallow wells in the gravel is a little harder and contains more free carbon dioxide and iron. The comparative value of the two sources of supply is, of course, determined in part by the quantity of chemicals required to treat the waters, in order to produce a water of good quality. Without treatment both of the well waters are objectionably hard and contain sufficient iron to make them of unsatisfactory quality for domestic use.

The analyses indicate that 2.11 lbs. of calcium oxide, or lime, are required to remove the temporary hardness from 1,000 gals. of the deep well water obtained from the rock; and that 0.18 lb. of sodium carbonate is required to remove the incrustants, or permanent hardness, from 1,000 gals. of this water. The cost of the lime is $1.50 per ton and of the sodium carbonate is $20 per ton. The commercial lime as purchased at Owensboro probably contains not more than 90 per cent calcium oxide. On this basis, the cost of lime required to soften completely 1,000,000 gals. of the deep well water would be $5.35. Commercial sodium carbonate generally contains nearly 100 per cent sodium carbonate. The cost of sodium carbonate required to remove completely the permanent hardness from 1,000,000 gals. of this water would be $1.80. In the same way the cost of lime required to soften completely 1,000,000 gals. of water from the gravel or the shallow wells would be $5.92, and of the sodium carbonate per 1,000,000 gals. would be $1.38.

TOTAL COST OF POWER AND CHEMICALS REQUIRED TO OBTAIN WATER FROM THE WELL SYSTEMS AT OWENSBORO

In the preceding discussion there has been shown the cost of power required to lift the water from each well system, also the cost of chemicals required to treat each of the well waters. These costs, together with the total costs per 1,000,000 gals., are shown in the following tabulation

COST OF 1,000,000 GALS. OF WELL WATER.

	Deep Wells.	Shallow Wells.
Cost of power	$13.70	$ 9.42
Cost of lime	5.35	5.92
Cost of soda	1.80	1.33
Total cost per 1,000,000 gals.	$20.85	$16.67

The cost of power required to drive the well pumps would be materially reduced were the pumps operated at a slower speed. The speed of the well pumps should be reduced to not more than 20 r. p. m. At this speed the pumps should be able to furnish a constant supply of at least 500,000 gals. daily. The theoretical capacity of each pump is 6.77 gals. per stroke. At a speed of 40 r. p. m. there should be produced 271 gals. per minute per well. As a matter of fact but 160 gals. are produced. This indicates that the well pumps are operated at an excessive speed. Under best operating conditions the cost of power would be about $7 per 1,000,000 gals. and the total cost of power and chemicals $14.25 per 1,000,000 gals.

Assuming that the present average daily consumption is 1,100,000 gals., the annual cost of power and chemicals, using water from the deep wells exclusively, would be $10,654, and using water from the gravel wells would be $7,282. These figures show that there would be a reduction in the operating costs of over $3,000 annually by obtaining all of the water used from the upper or gravel strata.

Construction Plant and Methods Employed in Reconcreting the Lining of the Eden Park Reservoir of the Cincinnati Water Works.

Contributed by Kenneth C. Crain, Cincinnati, O.

(The design of the new lining for this reservoir, consisting of broken stone, asphaltic felt and concrete, was described and illustrated in an article published in this journal of June 18, 1913.—Editors).

The work of relining with concrete the west basin of the Eden Park reservoir of the Cincinnati water works is now nearing completion. The Neeld Construction Co., of Pittsburgh, is the contractor. The work was undertaken last July under a 152-day limit, with a subsequent extension of 47 days, but this time, as is evident, proved inadequate.

The principal cause of the delay was the difficulty experienced in removing the old concrete, which was laid in 1876, and which averaged 18 ins. in thickness. In some parts it could be handled with pick and shovel, but for the most part its removal required the use of dynamite, and this while effective, was somewhat slow. The method used was, starting from the top, to drill holes under the concrete at varying distances apart, and as deep, or long, as 6 ft. The charges varied from half a stick to a stick to each hole.

The old concrete was left in the bottom of the basin, after a thorough testing to develop any possible weak spots. This was done by using a 14-ton roller, which traversed every portion of the basin without causing any yielding. It was assumed that this test gave an ample factor of safety for the greatest possible load of water, the maximum depth of water under service conditions being 30 ft.

The object of relining the basin was to prevent seepage of the water contained in the reservoir which, it is stated by the city authorities, sometimes amounted to as much as 2,000,000 gals. a day from the two basins. The capacity of the west basin, on which the work is now going on, is 50,736,000 gals., and that of the east basin, which is to be similarly treated, is 41,097,000 gals., indicating a daily loss by seepage of nearly 2 per cent of the total capacity.

The new lining, therefore, is designed to be as nearly waterproof and watertight as it is possible to make it. For this purpose, therefore, it is constructed under the following specifications, in brief: Graded, as far as necessary to secure a smooth surface on the slopes and a course of 6 ins. of crushed rock, from 1 to 3 ins. in greatest dimension, applied and tamped solid. Upon this is laid a preparation of heavy felt, impregnated with asphaltum, and about ¼-in. thick, made by the Barber Asphalt Co. This waterproofing comes in strips 15 ft. long and 4 ft. wide, and is laid with 6-in. laps, cemented together with pitch.

Upon this is placed a 6-in. slab of concrete, in 15-ft. squares, a 1-2-4 dry mix being used, the aggregate being crushed rock and old concrete, taken from the reservoir itself, 1 in. in greatest dimension and smaller. This new lining, it is believed, will make the basin absolutely watertight, thus preventing any further seepage of consequence. In order to take care of such surface waters as may find their way down under the felt, from the sides, however, 10-in. tile drains are provided. The lines of tile are placed under the bottom course of crushed rock, and led to a sewer outlet in the center of the basin.

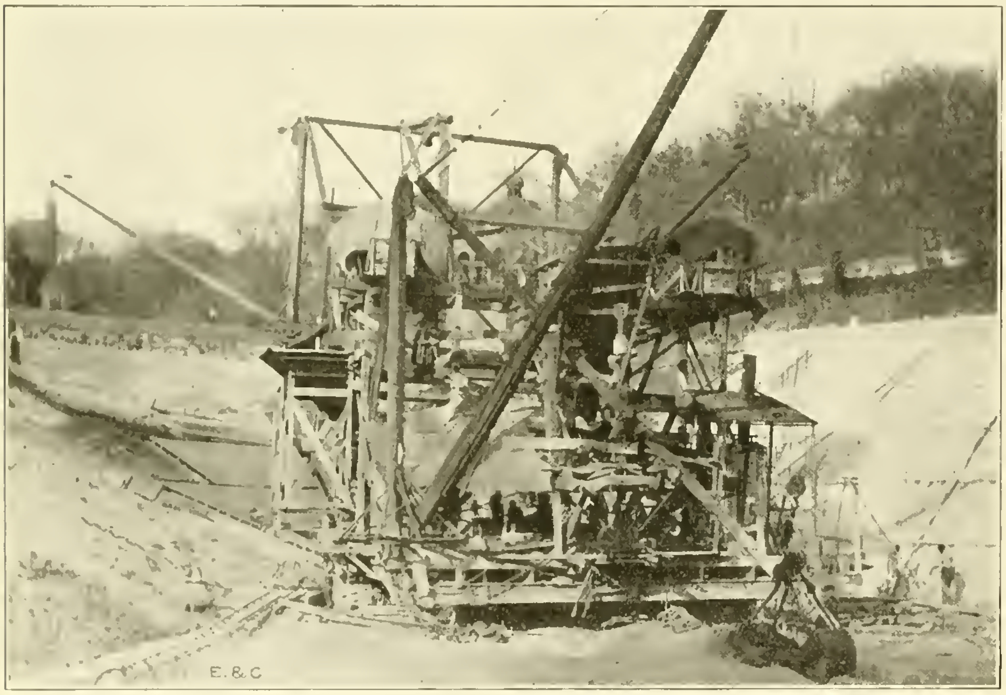


Fig. 1. View of Traveler for Transporting Concreting Equipment for Work on Slopes of Eden Park Reservoir.

A general view of the work is shown in Fig. 1.

The placing of the concrete in the bottom of the basin has been a simple matter. After the crushed rock layer is rolled and the felt placed upon it and allowed to become fixed, the mixture is wheeled to place in two-wheeled steel hand-carts. Fifteen-foot steel plates, ¼ in. thick and about 7 ins. wide, wired together, are used as forms. The thickness of these forms provide space between each square, after their removal, for a pitched joint A separate ¼-cu. yd. mixer takes care of this part of the work. It is operated by a 10-HP. motor.

The 85-ft. slopes, however, have presented greater difficulties in every phase of the work. The necessary re-grading, after the removal of the old concrete with dynamite, could not be done with a roller or any other of the usual equipment, on account of the steep pitch, running as high as 15° in many places; and this, therefore, was necessarily pick-and-shovel work. Likewise, the work of settling the crushed-rock bottom course had to be done by hand-tamping.

The placing of the concrete on the slopes, probably presented the most difficult problem of the entire job, not as being of a complicated nature, but as offering obstacles to speed The concreting plant finally evolved by the company consisted of a single traveler, bearing all of the washing, handling, conveying, crushing and mixing equipment This traveler is illustrated herewith in Fig. 2.

The crusher on this traveler was used only as long as the old material held out, crushed rock being hauled to the job since that time. A clam-shell bucket, operated from a boom by an engine on the traveler, lifts sand and crushed rock from the piles alongside the traveler to an upper level, from which they are chuted to the mixer as required. A ½-cu. yd. mixer is operated on this part of the equipment, dumping into a bucket which is lifted to the proper point on the slope by a boom working on a frame extended from the traveler

The concrete, poured from this bucket, is placed to the desired thickness by men with hand-tools, and, being a stiff mix, has given little trouble from sliding after placing. At the bottom of the slopes, however, when the concrete is laid on the slope before the bottom concrete has been placed, heavy timbers are used to prevent any movement of the material.

The work required 6,220 cu. yds. of concrete, and about an equal amount of crushed rock for the bottom course. The contract price of the work was about $102,000. The work will probably be completed in a short time. The arrangements for similar work on the east basin have not been made, but this contract will probably be let shortly after water is turned into the section of the reservoir on which work is now going on.

Results of Boston Tests and Studies of High Pressure, Cast Iron Bell and Spigot Joints, and of Jointing Materials.

Preparatory to the selection of the kind of pipe and the design of joint for use in the high pressure fire system of Boston, Mass., the Public Works Department of that city made a study of the performance of pipe line materials adopted for high pressure fire protection distribution systems in other cities, and made a set of carefully planned and executed tests on high pressure bell and spigot joints caulked with lead and lead alloy. The results of these studies and tests were presented by Mr. Clarence Goldsmith, of the engineering staff of the department, in his recent paper before the New England Water Works Association. The paper contains much original and valuable information and is published herewith in full:

Cast iron bell and spigot pipe has been used for the distribution of water in this country since early in the last century. The pipe of that period was cast in 8-ft. lengths without groove in the bell or bead on the spigot, and the joint was made up with lead in much the same way that it is today, but in some cases wooden staves were used instead of lead. This pipe was of comparatively small size, and the pressures to which it was subjected were in most cases less than 50 lbs. In the course of time larger pipes were required and higher pressures utilized, and about the year 1850 the casting of grooves in the bells and beads on the spigots was begun. The grooves gave the lead joint greater resistance against blowing out, and the bead facilitated the centering of the pipe, helped to hold the yarn, and contributed something to the holding power of the joint. Joints such as described have been adopted by the New England Water Works and American Water Works Associations in their standard specifications, and are now universally used on cast iron bell and spigot pipe laid for domestic distribution systems. Where bends, offsets, caps, plugs, blow-off valves and hydrants are set on the line it is frequently the practice to tie them into the line with rods passed either through lugs cast on the pipe, gates and fittings, or through wrought-iron bands bolted around the pipe and fittings, or else to use set screws tapped through bosses cast on the bells and set down onto the spigot end. In many cases, however, masonry abutments are built up against the line and fittings to resist the thrusts caused by irregularities in alignment and unbalanced pressure on caps and plugs.

With the advent of larger sizes of pipe, operated under continually increasing pressures, steel pipe was, in the latter part of the nineteenth century, utilized to a large extent to meet the then more exacting requirements. The life of steel pipe has in many cases been short, due largely to the lack of a durable protective coating, and in a measure to electrolytic action. Hence, when the question of a suitable pipe material came up in connection with the design of the earlier high pressure fire service systems, there was considerable diversity of opinion. In 1897 the city of Buffalo decided to use steel pipe and laid a mile of such pipe with threaded joints, but additions in 1904 and 1906 to this installation were of cast iron. On the other hand, in 1898 the city of Boston used cast iron pipe with a double groove in the bell for a high pressure salt water main of about one mile in length.

The first high pressure fire service system covering any considerable area was installed in Philadelphia in 1903, and although the original designs contemplated the use of steel pipe it was finally decided to use cast iron, and a flanged joint was designed for this special service. In 1909 it was decided to extend the system largely and "Universal" cast iron pipe was used for this work.

Cast iron bell and spigot pipe has been used exclusively in the construction of the high pressure fire service system of New York. The first pipe laid had double semi-circular grooves in the bell and on the spigot, and lugs or rings for wrought iron bands were cast on all pipes and fittings set where the alignment of the pipe was changed, and on branches and dead ends, thus providing for the tying of the line together with rods or bolts and nuts. In later installations the dimensions of the grooves were modified, but the practice of tying the lines together is still followed.

The city of San Francisco was the next city to undertake the construction of an extensive high pressure distribution system. Most careful and complete investigations were made before cast iron was adopted as the material for the pipe lines. Because of the possible trying and exacting conditions which might be imposed upon the distribution system by an earthquake, T. W. Ransom, M. Am. Soc. M. E. and consulting engineer of the board of public works, conducted an extended series of experiments, under the direction of City Engineer Marsden Manson, M. Am. Soc. C. E., on a large number of lead joints of varied design The results of these experiments led to the development of a joint which was adopted for the work and is producing excellent results The holding power of this lead joint, however was not sufficient to render unnecessary the use of lugs and rods on dead-ends and on fittings out of alignment

After careful study of the question of material for the pipes of the high pressure distribution system, the city of Boston has adopted cast iron as unquestionably the best for the purpose. In designing pipe for such service the engineer is largely guided by the results of previous practice, and as the superiority of the joint adopted by San Francisco had been demonstrated, both experimentally and practically, this type of joint was also adopted by Boston. The use of lugs and rods was, however, considered open to criticism and serious objection, it being practically impossible to so adjust the rods in sets of four or six that each would take even an approximate share of its load. The rods, too, are subject to rapid deterioration when buried in earth of the kind found in the greater part of the area in which it is proposed to lay the pipe. The cost of each joint made up with lugs and rods exceeds that of a plain lead joint by approximately $20 for 20-in. pipe, $15 for 16-in., $7 for 12-in. and $4 for 8-in. These objections suggested the development of a more reliable and permanent method of holding the pipe line in place: but an even more serious objection presented itself to the use of the rod and lug construction in Boston. On account of the great congestion of existing underground structures it is of the utmost importance to economize in space by reducing the clearances to a minimum. The minimum clearances required for pipe with lugs is 35½ ins. for 20-in. pipe, 29½ ins. for 16-in., 24¼ ins. for 12-in. and 18¾ ins. for 8-in., while without lugs these clearances could be reduced 6, 5, 5 and 4½ ins., respectively. The advantages of the latter construction are therefore obvious. A cross section of the adopted joint is shown in Fig. 1.

Several methods of overcoming the difficulties which presented themselves were studied. First, the use of set screws in bosses cast on the bells was considered, but their holding power, when of reasonable size, would not be sufficient; and other objections were the difficulty of making them watertight, the expense of drilling and tapping the holes, and the work involved in adjusting the set screws.

Interlocking wedges cast on the spigot and in the bells were next designed, but were not considered feasible, as the adjustment of the pipe in the ditch would be difficult, and there would be practically no flexibility of the line in case of settlement.

JOINT MATERIAL.

Experiments showed that the failure of double-groove joints of the design adopted was due to the flowing of the joint material, rather than to the shearing of this material in the groove of the bell as in the case of single-groove joints. It was clear, therefore, that the problem would be satisfactorily solved if a material could be found which would offer a much greater resistance to flowing than lead and could be poured and driven with moderate facility, and yet would not be so rigid as to prevent a sufficient movement in the joint to allow for settlement of the pipe line.

The materials available for an alloy of the character required are as follows:

Material	Melting point.	Tensile strength, lbs.
Lead	600° F.	1,600 to 2,400
Tin	450° F.	3,500
Bismuth	500° F.	6,400
Zinc	780 F.	5,000 to 6,000
Antimony	1,150° F.	13,000

To make the various joints for testing, 20-in caps and plugs with double grooves of the design adopted, cast in the bell and turned on the spigot were used. The depth of the bell in all cases was 4½ ins. and the joints were made up by centering the plug in the cap and yarning the joint until the face of the yarn, which was driven home with a hammer, was 3½ ins. from the face of the bell. The plug was then tipped at an angle of about 45°, a jointer adjusted, and the joint material poured. A round jointer provides for a sufficient lip of lead on the outside to permit of the joint's being flush when driven. If a flat jointer is used, its inner side next the joint should be beveled at 45° for a distance away from the pipe equal to the thickness of the joint in order to provide sufficient lip. The thickness of the joints tested varied from 0.35 to 0.75 in. The caps were tapped with two ¼-in. holes, one to receive the connection from a small pump capable of raising the pressure to 3,000 lbs., the other to provide a vent for air.

In the first twelve tests observations to determine the movement of the joint were made with a rule and readings were taken to 1/16 in. at two points on the plug. The pressure was raised in increments of 100 lbs. and maintained constant for intervals of, generally, five minutes. In this series of tests two joints were made up with Omaha lead, four with 96 per cent lead and 4 per cent tin, one with 94 per cent lead and 6 per cent tin, two with 97 per cent lead and 3 per cent tin, two with 98 per cent lead and 2 per cent tin, and one with lead wool hand driven. The data obtained showed that an alloy of lead and tin made a joint which offered a greater resistance to the unbalanced pressure between the cap and plug than Omaha lead; that as the proportion of tin in the alloy is increased the resistance of the joint is increased; that lead wool offers a greater resistance than Omaha lead, but not so great as any of the alloys used; that the joint did not reach its maximum power of resistance until the plug had moved some little distance out of the cap; that the alloy could be poured with about the same facility as lead, but that the driving of the joint required more time as the quantity of tin was increased; also that the joints when driven are watertight, but when a joint moved a slight leakage was observed, which, however, disappeared before the joint had moved out far enough to develop its maximum strength. The method of making the observations, however, was not sufficiently refined to determine exactly when the joint attained its greatest strength, and the intervals during which pressures were maintained were not long enough to assure equilibrium of the joint.

A series of three tests (Nos. 14 to 16) was run in which measurements to 1/100 in. were taken by dividers at four points at the ends of diameters at right angles to each other, and the time interval increased in the last test to 55 minutes. In two of these tests an attempt was made to reproduce the condition which exists when the alignment of the pipe is changed in the trench, that is, in which one side is home, and the other side is in one case ½ in. and in the other 1 in. from being home, corresponding to throws of 3 ins. and 6 ins. respectively, in the trench for a full length of the size of pipe being tested. This condition did not appear to affect the resistive power of the joint.

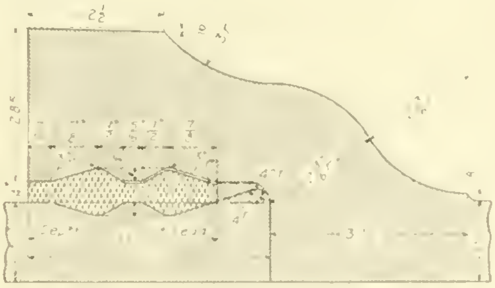


Fig. 1. Section of 20-in. Bell and Spigot Joint for Proposed High Pressure Distribution System, Boston, Mass.

These preliminary tests having proved that it was possible to make a joint strong enough to permit the rods to be dispensed with, a series of 13 tests was next run under more reliable and accurate methods of measuring the joint movement, and with pressures maintained for a time sufficient to assure that the joint was in equilibrium. Three of these tests were made with Omaha lead, one with lead wool, three with an alloy of lead and tin, three with lead and antimony, and three with lead, tin and antimony.

Although the results obtained with a 4 per cent tin alloy were fairly satisfactory, the ultimate resistance of the joint was only about 500 lbs., and the cost of tin is about 50 cts. a pound, while antimony costs but 10 cts. a pound and reduces much more than does tin the tendency of the lead to flow. In melting the lead and antimony to form the alloy it was found desirable to melt the antimony in a separate pot and add it to the molten lead, for if they were melted in the same pot the high temperature required to melt the antimony would cause a burning of the lead. The temperature required to melt the metals after they are synthetized is 900° F., or approximately half-way between the temperatures required to melt them separately, and when heated to a proper temperature for pouring the oxidation of the alloy appears to be less than that of lead.

In the tests of joints made up with 96 per cent lead and 4 per cent antimony a number of small leaks developed between the pipe and the joint material, even when the joint was carefully driven, showing that the ring of alloy does not contract sufficiently around the pipe to make a watertight joint, whereas lead or a lead-tin alloy assures watertightness. For this reason it was not practical to adopt this lead-antimony alloy, although its holding power was satisfactory.

The three tests made on the alloy composed of 96 per cent lead, 2 per cent antimony and 2 per cent tin, showed that the alloy can be easily melted and poured, requires only about one-third more time to drive than lead, and meets the requirements in other respects. This alloy was therefore adopted as the best material for the joint, and the joints thus made were found capable of resisting the unbalanced force developed by the pressure of 700 lbs. per square inch on the 20-in plug, amounting to 312,000 lbs., which, since the circumference of the joint is 73.6 ins., is equivalent to a resistance of 4,240 lbs. per linear inch. The pressures which can be carried on plugs of other sizes may be readily calculated and are shown in Table I together with the calculated ultimate holding power of lead joints tied together with rods.

It will be seen from Table I that the strength of the alloy joint is fully as great as the combined strength of the lead joint and rod for sizes of pipe up to and including 20-in. Although 20-in. pipe is to be the largest in the proposed system, 24-in. has been included in the table because pipe of this size has been used in some installations. A study of the plots of the various tests shows that the joint does not attain its maximum resistance to the internal pressure until the plug has moved out about 0.15 in. This is due to the fact that the alloy contracts in cooling, and as the calking does not affect the material in the inner groove this part of the joint does not offer its resistance until it is brought into bearing by the outward movement of the spigot end. During the progress of the several tests no leakage was observed until a pressure of 500 lbs. per square inch was reached, and the leakage at

TABLE I.—COMPUTED STRENGTH OF JOINTS SUBJECT TO HIGH PRESSURE.

Nominal diameter of pipe, ins.	Number of rods.	Diameter of rods, ins.	Strength of sets of rods, in pounds.	Resistance of plain lead joint, in pounds.	Combined strength of lead joint and rods, in pounds.	Resistance of lead-antimony-tin joints, in pounds.	Maximum allowable pressure for lead joints, in pounds.	Maximum allowable pressure, in pounds.
8	4	1	82,800	17,400	100,200	133,100	[illegible]	1,550
12	4	1¼	86,700	68,700	151,400	192,900	[illegible]	1,100
16	6	1¼	128,600	89,800	218,400	252,300	[illegible]	900
20	6	1½	186,500	111,200	297,700	312,000	250	700
24	6	1¾	251,100	133,300	384,400	374,200	200	600

this pressure, which was small, ceased before the joints had attained their ultimate strength.

In order to make a more complete and practical test of this joint material, a 20-in. line, consisting of a length of pipe, four 1/16 bends, and a second length of pipe, was set up on blocking, and the ends were closed with a cap and plug. The joints were made up with yarn and an alloy composed of 96 per cent lead, 2 per cent antimony and 2 per cent tin, except that joint No. 6 was poured with a lead-tin alloy. All joints were driven with dog tools and finished with hand tools. The movement of the line while under pressure was recorded in the following manner:

A ¼-in. hole about 1 in. deep was drilled in the top of each of the seven bells, and a wooden pin, into which a sewing needle, pointing up, was inserted, was set in the hole. A stake was driven on either side of the pipe opposite each joint, so that a cross-piece extended over the pipe would be at right angles to the axis of the pipe. A small board upon which cross-section paper was mounted was so hinged to each cross-piece that the needle in the wooden pin beneath it would just pierce the paper when the board was in its horizontal position. Holes were so drilled and tapped in the line that it could be filled and emptied, and provision was made to vent the line of air and to attach a pressure pump. After the line was filled with water a record was taken of the position of each bell by swinging down the boards on which the cross-section paper was mounted until a hole was pricked by the needle point. The pump was then attached and the pressure was raised to 300 lbs. per square inch and maintained for four hours. During the last two hours of this period the line was in equilibrium and no leakage developed. The pressure was next raised to 400 lbs. per square inch and maintained for eight hours, and during the last four hours the entire line was in equilibrium, and no leakage was apparent. The pressure was then raised to 500 lbs. per square inch, and joint No. 6, which had been made up with an alloy of lead and tin, did not come into equilibrium, but continued to draw out of the bell, and when it had drawn out 1.9 ins. the test was discontinued. This joint was melted out and a new joint of lead-antimony-tin alloy was made, and the test continued. The pressure was again raised to 500 lbs. per square inch and maintained for three hours, then raised to 600 lbs. per square inch and maintained for six hours. During the last five hours the pipe line was in equilibrium, but several of the joints sweat during the entire time. The leakage on joint No. 1 was the greatest, and equalled six fluid ounces per hour, which is equivalent to a leakage of ¼ gal. per linear foot of joint for 24 hours. An attempt was then made to raise the pressure to 700 lbs. per square inch, but the capacity of the pump, which had a displacement of only 1/7 of a fluid ounce per stroke, was not sufficient. A second pump of larger capacity was secured, and with the two pumps in service a pressure of 700 lbs. per square inch was maintained for 15 minutes, when one of the bends burst. During this period the leakage increased but slightly, and no observations were taken.

The results obtained from the foregoing experiments showed that the lead-tin-antimony alloy would amply fulfill the requirements for making joints in cast iron bell and spigot pipes with grooves of the design adopted. Inasmuch as it has frequently been necessary in the construction of ordinary low pressure pipe lines for the distribution of domestic supply to tie the various fittings into the line with rods or set screws, or sometimes to build up abutments to secure them, it is most desirable to ascertain the effect of an alloy in the making up of joints in ordinary cast iron pipe. Two 16-in joints of Boston Water Works design were, therefore made up, one with lead and one with an alloy of 96 per cent lead and 4 per cent antimony. The lead joint withstood a pressure of 300 lbs. per square inch, while the lead-antimony joint withstood a pressure of 400 lbs. per square inch, showing an increase of one-third over and above the holding power of the lead joint

Water Consumption in Indiana.

Contributed by J. A. Craven, C. E., Sanitary Engineer, Indiana State Board of Health.

In portions of Indiana the ground water level has lowered from 5 to 20 ft. within the memory of the older citizens of the state. The cities of Muncie and Indianapolis have changed from a well to a filtered surface supply. Some well water is used in conjunction with the filtered water at both these places but wells alone were insufficient. Much water is wasted in Indiana as elsewhere. The greater part of this waste comes from a lack of knowledge on the part of the average citizen, and shows the need of education along the line of water conservation.

Actual figures on water consumption have been obtained from 72 cities and towns in Indiana, and Table I gives the number of cities and towns in four divisions according to the number of inhabitants, the average daily consumption per capita, the variations in consumption and the classification as to private and municipal plants.

TABLE I.—WATER CONSUMPTION IN INDIANA CITIES.

Population.	Number of cities and towns.	Consumption, gals. per capita.	Variation in consumption.	Kind of plants.
Under 5,000....	29	81.2	10-350	14 Private 15 Municipal
5,000-20,000....	27	96.7	26-240	15 Private 12 Municipal
20,000-50,000....	11	126.6	45-320	7 Private 4 Municipal
Over 50,000.....	5	90.6	75-140	2 Private 3 Municipal

In the first divison, those under 5,000 inhabitants, 23 cities depend upon wells, two upon raw river water, one upon filtered river water and three upon springs.

The average of the 14 private plants is 67 gals. per capita and of the 15 municipal plants 95 gals. The one private plant filtering the water averages 55 gals. per day per capita. Two of the municipal plants using river water pump 139 and 217 gals., respectively. It is easy to see how such a supply could be wasted, but anyone acquainted with the well water situation can hardly conceive of another town pumping 350 gals. per capita daily from such a source.

In the next group, those having a population of 5,000 to 20,000, 27 cities are represented. In this section we have 15 private and 12 municipal plants. The sources of supply in this group are more varied than in the first section. Seventeen use well supplies, three raw river water, three filtered river water, one lake water, one a mixture of river and well water, one lake and well water and another surface and spring water. The average daily consumption of the 27 cities is 96.7 gals., that of the 15 private plants being 100 gals., and of the municipal plant 93 gals.

One private plant using lake water, untreated, furnishes 240 gals. per capita, and two others pumping untreated river water use 100 and 118 gals., respectively. Two of them filtering water use 111 and 126 gals. per day per capita, respectively.

Not one of the plants pumping well water use an extravagant amount, according to American figures, their average being 79 gals.

In the group whose population ranges from 20,000 to 50,000, eleven cities are represented by seven private and four municipal plants. The average daily consumption per capita is 126.6 gals. per capita, that of the municipal plants being 177 gals., and private plants 102 gals. The average figures for the private plants is greatly increased by the consumption of 240 gals. at one city. Here lake water, an unlimited supply, is used and the only expense to be considered is the cost of pumping. The average of the six remaining cities is only 79 gals. per capita. Three of the six use well water, one lake, one river and one filtered water.

It is only fair to say the high average at the municipal plants is because of per capita consumption of 350 gals. at a city using lake water and 228 gals. where a raw water is used. One of the plants where the water is filtered pumps 61 gals. per capita and the fourth using well water 68 gals. per capita.

In the fourth division, those over 50,000, two are private and three municipal plants. The per capita consumption is 90.6 gals. One private plant filtering the water uses 78 gals. per capita, while at the second where a filtered and well supply is furnished the per capita consumption is 82 gals. Two of the municipal plants usng wells pump 75 and 78 gals. per capita. The third municipal plant filters the supply and the average daily consumption is 140 gals. per capita.

It is surprising when questioning many water works engineers and officials, to find how little they know about the amount of water used, and although some of the figures submitted by the companies may be far from the truth, it is believed that the average is fairly representative. While this might be expected in a municipal plant where politics play a large part in the appointments, yet the writer has found the same state of affairs in some private plants. Some say frankly they don't know, others hesitate for a while and then make a wild guess in estimating their pumpage. Some know that their pumps are run about eight hours a day and others that about half a standpipe is used each day. A few cities and towns were omitted from the table. At one of them the average daily consumption per capita based on figures submitted was a little over one gallon. The low figures, 10 and 26 gals. in the first two divisons, given as the minimum variation, may be accounted for by the fact that the supply is only fit to use for sprinkling purposes, and the consumption is necessarily low.

As compared to the other American cities, the average of the 72 cties and towns, 94.6 gals. is quite conservative. The average of 19 cities was 130 gals., the variation being from 37 to 250 gals. In many of the cities more than half of the services were metered. Although the consumption per capita is much below the average American rate, yet as compared with that of Europe, it is high. The average for eight of the largest European cites is 34 gals. per capita per day. Allen Hazen in his book on Clean Water says. "Probably the whole cost of supplying water in America is from a third to a half greater than it would be with the reasonable suppression of waste."

Not long ago a small town requested the advice of the State Board of Health in regard to an increased supply. During the course of an inspection of the present system, a leak was observed at an outside hydrant, the measurement of which revealed a loss of 254 gals. per day. An inspection of a few more buildings revealed three taps wasting altogether 864 gals. per day if open for the 24-hour period. It was said that these taps could usually be found running. Although time was not taken for a more complete survey of the waste, information concerning many more leaks was given. It is believed that if the present waste was curbed an additional supply would not be needed at this time and they were advised to this end.

It is plain, to one acquainted with Indiana conditions, that a great saving can be effected by a more careful supervision of the distribution system. To this end, a metering of the entire system would be the most effective, but the question of the initial cost of meterage plays a great part at the smaller plants.

Similar conditions can undoubtedly be found in other states as well, and it is believed that educating the public to appreciate the seriousness of the situations at many places would assist materially in the conservation of our water supplies.

Engineering and Contracting

Devoted to the Economics of Civil Engineering Design
and to Methods and Cost of Construction

Volume XLI. CHICAGO, ILL., MAY 20, 1914. Number 20.

Three Interesting Examples of Dam Design.

There are illustrated in this issue a low head water power dam in Indiana, an irrigation water diversion dam in California and a high head water power dam in Mexico. These dams are all of American design and each design presents some feature which is worthy of attention by the dam builder. To aid the reader pressed for time the several features had in mind may be pointed out, but the discriptions and drawing should be studied for details.

The Mexican dam and the dam for the Orland irrigation works in California are similar and notable for their very thin section of reinforced concrete. Information of the method of computing the Reclamation Service dam is not at hand, but the Mexican dam, as stated in the article, was computed by the simple formula for cylinders subjected to external pressure. This practice is that followed in designing several recent arch dams of magnitude by Australian engineers. The success of these Australian dams is in fact one of the arguments offered by the designer in defense of his method of designing. Since the analysts of dam stresses are apt to have something to say in answer to his implied criticism of their processes we quote the designer's arguments in full:

The formula does not satisfy all the conditions, mathematically, for the dam is neither a complete cylinder with free ends nor is it everywhere free to expand and contract; on the contrary, it is a segment connected at its bottom and sides with the immovable, but not the less elastic, country rock; and it is not a thin cylinder, but one of appreciable thickness. The problem is impossible of exact mathematical solution, for, however approached, its treatment is based on assumptions. The break of continuity in both forms and material at the line of contact between the artificial structure and the natural rock, and the unknown value of the element of deformation in the natural rock, create uncertainties which, in themselves, render a mahematical demonstration based solely on absolute facts an impossibility, for the facts are not and cannot be known. Such elements of uncertainty are not confined to arch dams alone: they are equally prevalent with gravity dams. The researches and experiments of Pearson, Otley, Brightmore, Baker, and others have shown the accepted middle-third method of gravity section design to fall short of being absolutely correct. Nevertheless, the fact remains that dams designed on that theory, within certain recognized limitations as to intensity of stress, have been and continue to be successful. Similarly, the many Australian arch dams designed on the simple cylinder theory have been and continue to be successful.

Whatever the unknown and undeterminable stresses may be, particularly those around the foundations, the adopted formulas, empirical though they may be, have given and do give safe results, and, in the present status of knowledge as to the actual stresses in dams, their use must be considered to be more conservative than that of formulas mathematically correct but based on assumptions of unknown conditions. The successful practice of engineering is one of applied science, rather than of pure science, and the pure mathematical formula, without some modifying coefficient derived from practice, is a rarity.

The views expressed by Dr. W. C. Unwin, Past President of the Institution of Civil Engineers, in a discussion on the subject of stresses in dams, are very pertinent.

"It had now been shown that it was hopeless to attack the dam problem by pure mathematics. He did not think there was any theory on which an engineer relied in designing a bridge, a roof, or a retaining wall, to which mathematical objections could not be raised of the kind that had been raised against the ordinary theory of dams. The state of stress in a plate around a rivet was as complex as that in the foundation of a dam, and was as little susceptible of mathematical treatment; and yet engineers were not afraid of making riveted joints."

It may be said, with equal truth, that engineers will not be afraid to continue building gravity dams on the middle-third theory, or arch dams on the cylinder theory, and no disaster will follow, always provided that there is due recognition of the conditions of the foundation and abutments which nature provides.

The futility of attempting a pure mathematical determination of the stresses in a dam is well exemplified by the measurements of the actual deflections of an arch dam at Barren Jack, New South Wales.

The structure is of concrete with a light vertical reinforcement (against temperature stresses) of 20-lb. steel rails, 10 ft. from center to center and 1 ft. from the face. As the steel area is only 0.044% of the area of the concrete at the base, and 0.139% at the top of the dam, its presence is negligible, as far as bending stresses are concerned.

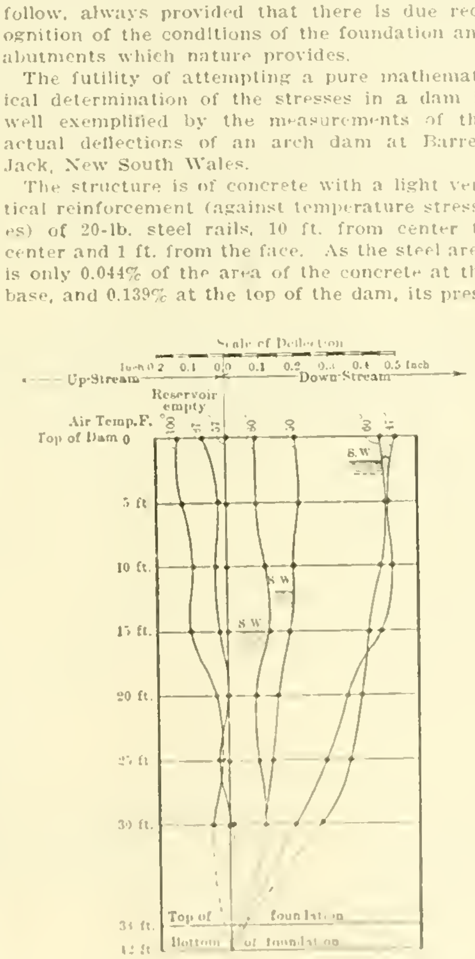


Fig. 1.

The deflections of the Barren Jack Dam, which are shown in the diagram, Fig. 1, were taken under varying conditions of temperature and water depth. The observations made when the reservoir was empty and the air temperature 57°, which approximates the closest of the observations to mean temperature, are drawn as a straight line, and the other observations are plotted in regard thereto. The curves drawn through the various observed points are brought for convenience to one common point of origin on the foundation base. This is not strictly correct, for it makes the impossible assumption that the base is absolutely rigid, and that the rock on which the dam stands is incapable of elastic deformation under the stresses transferred to it by the dam and by the weight of water supported by the rock bed immediately above the dam. Deformation of the rock undoubtedly takes place and such being the case, the arch action takes place in some unknown degree from the very base of the dam. That deformation of the bed-rock does take place to considerable depths was the opinion expressed by the late Sir Benjamin Baker, Hon. M. Am. Soc. C. E., in discussing the Coolgardie Water Supply.

Fig. 1 indicates that the bending stresses at the base are probably little if any greater than at higher points of the dam. Were the bending stresses at the bases of arch dams as great as some theories would lead us to think, many if not all the arch dams cited would have been fractured, and in particular the very thin dams of Bear Valley and the Upper Otay. That they have not been fractured is conclusive that the theories need amending.

The argument for the sufficiency of empirical formula in designing dams may be left as stated. The Reclamation Service dam has besides its three sections the unusual feature of a siphon spillway. Such spillways have been used in a number of cases on the New York Barge Canal and we believe on irrigation canal work in a few instances, but they are in general unusual practice and so demand notice. The drawings shown of the siphon spillway in another column are more complete than are easily available elsewhere.

Referring to the Mexican water power dam, the one practice demanding special mention is the provision for steam chambers in the gates and gate abutments to prevent ice obstruction. This feature deserves attention.

Some Design and Construction Features of the Des Moines River Viaduct.

The Chicago, Milwaukee & St. Paul Ry. has completed a double-track steel viaduct across the Des Moines River, near Madrid, Ia. This viaduct, the design and construction of which are fully covered in the bridges section of this issue, possesses a number of interesting features. It has a length of 2,394 ft. and a maximum height of about 130 ft., and consists principally of a series of 40-ft. tower spans and 70-ft. intermediate spans, with two 155-ft. deck truss spans over the channel of the river. The reinforced concrete floor for each track is separate construction, and consists of a slab, cast in sections, resting on the cover plates of the girder spans. To decrease the amount of field riveting the transverse girders at the top of the tower bents are shop riveted to short sections of the tower posts, which are spliced below the connecting gussets of these girders. The tops of the pedestals which support the tower posts are inclined, thus making their bearing surfaces normal to the axes of the columns. The abutting ends of the two adjacent truss spans over the main channel of the river rest on a rocker bent. To secure equal traction forces in the two shore towers which support the outside ends of these truss spans the west truss has both ends fixed, while the east truss has an expansion bearing at the rocker bent and a fixed bearing on the shore bent.

The pile foundations of the pedestals were jetted through about [illegible] ft. of sand and then driven from [illegible] the hammer; the water required for jetting the piles being applied through a [illegible]-in. pipe at a pressure of from [illegible] to 100 lb. per square inch. The concrete for all pedestal shafts was mixed in a continuous mixer, which required working at night for some of the largest pedestals. Most of the concrete was laid during the winter. Excellent results were obtained by heating the sand by means of steam pipes to about 100° F. and the water to about 150° F., it being found unnecessary to heat the broken stone. After being poured the pedestals were housed in, and live steam was turned on for about four days.

With the exception of the lower stories of the towers the viaduct was erected from the track level with a 50-ton derrick car, equipped with an 80-ft. boom and a 50-HP. engine. Where the ground was comparatively level the lower stories of the towers were erected with a mule traveler, equipped with two 60-ft. booms and two double-cylinder engines, this traveler being operated over an 18-ft. gauge track laid along the center line of the viaduct. The most satisfactory erection procedure was to erect each bent complete, then to set the girders and laterals, and lastly to place the longitudinal tower bracing. The truss spans were erected on a falsework trestle, supported on a pile foundation. Each story of this falsework was framed in units, the units for the lower stories being constructed on the pile caps and then lifted into place by the mule traveler. Those for the upper story were framed at one end of the bridge, and were placed by the derrick car as the work progressed. The construction of this viaduct required practically one year, ground being broken July 10, 1912, and the work completed July 12, 1913. It is of interest in connection with this viaduct to note the data on several other long viaducts given in this issue.

Some Operating Engineering Problems of Mountain Side Conduits.

An irrigation or water power "canal" in the mountainous west is frequently a complex combination of types of conduit for carrying water. Earth canal, elevated flume, tunnel and pipe line are usual in a single canal. Such a combination of structures plainly offers problems in design and construction which are full of variety. Design and construction furthermore do not conclude the engineer's work. A contributor in this issue presents interesting evidence that even during the first few years of operation, redesign and reconstruction may easily furnish engineering problems nearly as complex and as costly to solve as those of the original conduit work.

This article on the Arkansas Valley conduit to which we refer particularizes a danger to mountain side canals which is not commonly mentioned. Indeed a search of immediately available books and articles discloses no mention of it. Slides of earth or rock caused by cutting along the slope for canal or due to nature's continuous work of readjusting her mountain sides are not of the same kind. As described by the article, where an open canal comes up to the edge of a crossing gulch or arroyo and there has junction with a closed system which dips across the valley; the water coming down the canal finds in the bottom at some point back of the siphon intake a previous stratum which it follows down the slopes of the gulch and lubricates and loosens until a slide results. This article describes three slides due to this cause and also describes the methods employed to repair the damages, and all the repairs were expensive ones.

Another kind of siphon trouble which has not to our knowledge been previously recorded is that due to ice plugs. The curious formation and damaging results of this ice formation are sufficiently explained by the article. Another serious operating problem emphasized by this article is the handling of silt accumulations. The article describes two methods, one of which, the provision of a miniature hydraulic dredge for channel cleaning is unique. Concrete encasing rotting wood stave pipe is another unusual maintenance method described. Altogether the article gives a variety of reasons why, as its author says, there is something to do after a mountain side irrigation or water supply canal is built besides sitting on the bank and watching the water flow by.

Bridges

Design and Construction Features of the Des Moines River Viaduct near Madrid, Iowa.

Contributed by J. H. Prior, Assistant Chief Engineer, Illinois Public Utilities Commission, formerly Engineer of Design, Chicago, Milwaukee and St. Paul Ry.

The Chicago, Milwaukee & St. Paul Ry. has recently completed a steel viaduct across the Des Moines River valley, three miles west of Madrid, Ia. This bridge is only 17 miles south of and spans the same valley as the well known Boone viaduct of the Chicago & Northwestern Ry. and it ranks with other notable structures of this type.

The viaduct forms a part of the double-track reconstruction work being done by the Chicago, Milwaukee & St. Paul Ry. on its Chicago and Council Bluffs division in Iowa, and is on a new alignment extending from Madrid to Woodward, as is shown in Fig. 1. The present line between these two points crosses the Des Moines River about two miles southeast of the new line, and is a single-track line, 7.65 miles in length. The new site reduces the distance 2.13 miles and eliminates 790° of curvature and 197 ft. of rise and fall.

From the east the viaduct is approached by a double-track fill having a maximum height of about 100 ft. and a length of 3,500 ft. It contains approximately 1,300,000 cu. yds. of material. The bridge is on a grade of 0.66 per cent, rising toward the west, and is a double-track structure, with a total length of 2,391 ft. 2 ins. out to out of steel, and a maximum height above the ground of about 130 ft. The superstructure (see Fig. 2) consists, in general, of 40-ft. towers and 70-ft. intermediate girder spans. The portion which spans the main channel of the river consists of two 155-ft. riveted deck truss spans, supported by the adjacent tower bents and a single rocker bent in the middle of the channel.

The viaduct was designed for Cooper's E-55 loading. The dead load used consisted of the estimated weight of the structure and that of the reinforced concrete deck slabs and a ballasted floor, the weight of the latter being taken at 3,000 lbs. per linear foot of track. The structure was designed according to the specifications of the Chicago, Milwaukee & St. Paul Ry., which are similar to the general specifications for steel railway bridges of the American Railway Engineering Association.

THE SUBSTRUCTURE.

Borings showed that from the west abutment up to and including bent No. 27 there is a thick bed of stiff yellow clay at a depth of from 6 to 10 ft. Tests of this clay showed that it would safely carry a load of 3 tons per square foot, and the footings for the west abutment and for the pedestals up to and including those for bent No. 27 were designed for this unit load.

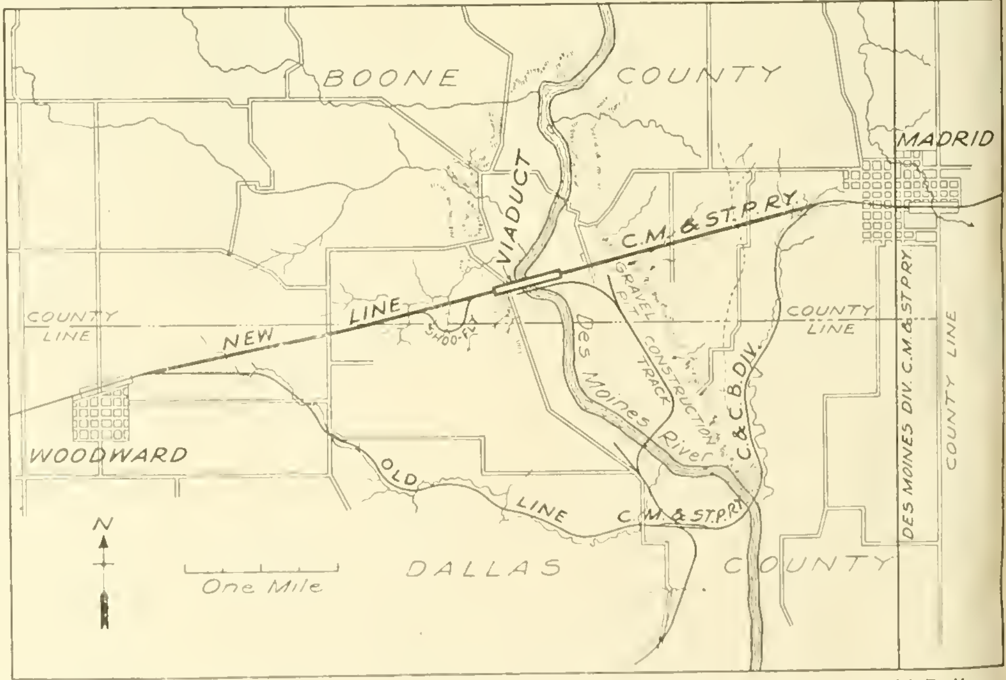
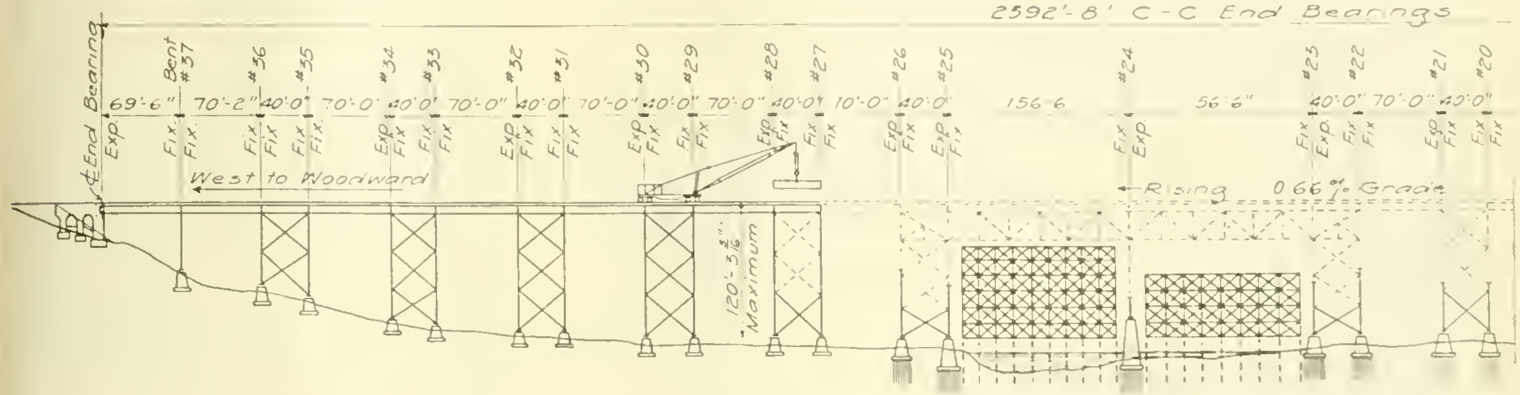


Fig. 1. Map Showing Location of Des Moines River Viaduct and of New and Old Railway Lines.

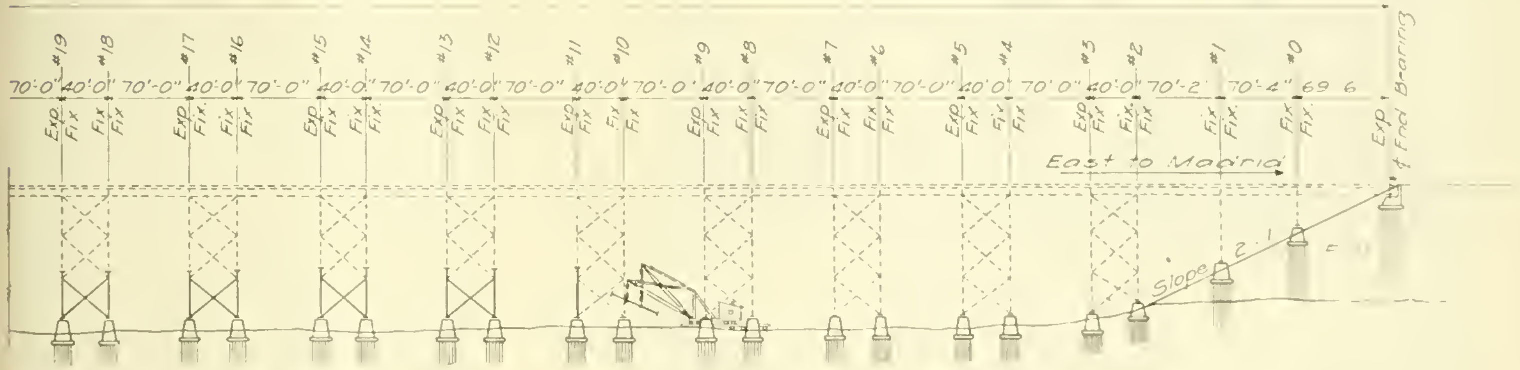
Figure 3(a) shows in outline the side and end elevations of the west abutment, and indicates the type of construction used.

From bent No. 26 to the east end of the viaduct the surface layer consists of sand, from 25 to 35 ft. in thickness, beneath which there is a stratum of hardpan. Pile foundations were used for the footings of these bents. On an average the piles were jetted 26 ft. through the sand and were driven from 12 to 14 ft. into the hardpan. The jetting was done with a 2-in. diameter pipe, a pressure of from 140 to 160 lbs. being maintained at the nozzle. The hardpan was penetrated with difficulty, the piles moving only about 1 in. under each blow of a 2,800-lb. drop-hammer, falling about 30 ft. The last inch of penetration required, on the average, three blows. The average pile load was taken at 18 tons.

The pedestals, except those in bents Nos. 0 and 1 (see Fig. 2), are 6 ft. square on top and slope toward the center line of the viaduct, making the bearing surface normal to the axis of the columns which have a batter of 1½ ins. in 12 ins. The sides of all pedestals are battered. Figure 3 (b) shows a typical downstream pedestal as built for bents Nos. 8 to 24, inclusive. The upstream pedestal is provided with a nose. Each pedestal has two 2-in. diameter anchor bolts, 10 ft. long, which

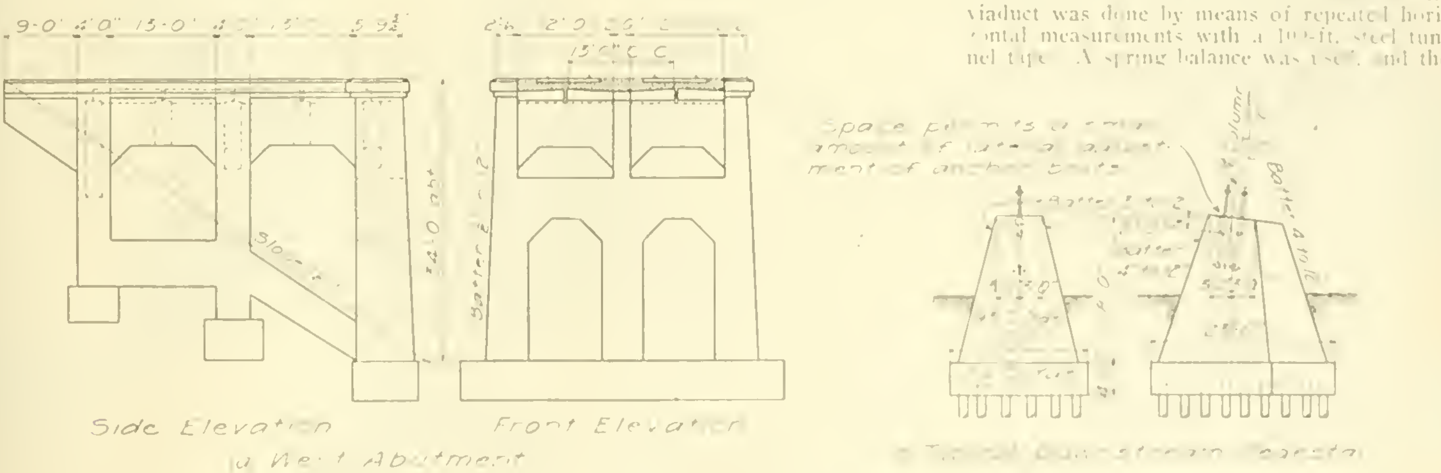


(a) West Part.



(b) East Part.

Fig. 2. Outline Drawing of Des Moines River Viaduct Showing General Dimensions and Methods Used in Erection.



Side Elevation Front Elevation

West Abutment

Typical Downstream Pedestals

Fig. 3. Elevations of West Abutment and Typical Downstream Pedestals of Des Moines River Viaduct.

project 3 ft., 9 ins. above the tops of the pedestals. These bolts pass through the bed plates and engage the lug angles which are riveted along the lower ends of the columns. The rods are anchored by means of spacers, the concrete being poured around them. To provide for variations in their positions box forms (extending 2 ft. below the top of the pedestals) were provided around each anchor bolt. The holes so formed were grouted after placing the bed plates. In addition, each bed plate was fastened to the masonry with eight 2-in. diameter fox bolts, each 2 ft., 6 ins. long, the holes for which were drilled after erecting the bed plates and columns. To provide for any lateral motion which may accompany the settlement of the fill, the pedestals in bents Nos. 0 and 1 were built with horizontal tops, 8x10 ft. in size, and the bed plates for these bents were provided with slotted holes. This permits the plates to be shifted and the superstructure to be adjusted in case the pedestals are pushed forward. Creosoted piles having length of from 40 to 45 ft., sustain these pedestals, the average load on the piles being 12 tons.

Both on account of the expense of an abutment built from the ground up and due to the uncertainty of its withstanding the earth pressure and the settlement of the fill, it was decided to use a small bank abutment at the east end until the fill had settled sufficiently to justify the construction of a more permanent type of abutment. The abutment rests on 60 ft. creosoted piles which carry an average load of 12 tons each. It consists of a thick reinforced concrete slab with a timber backwall. Bars are provided for a permanent concrete backwall which will be constructed when the embankment has thoroughly settled.

The concrete is a 1:3½:5 mix for the footings, and 1:3:4½ mix for the neatwork. An excellent grade of sand was found about ¼ mile east of the viaduct. The sand contained about 20 per cent of gravel and was used for this work. The location of this gravel pit is shown in Fig. 1. The stone used was a good grade of screened limestone, with maximum dimensions of from 1 to 2 ins. No crusher run stone was used. The water was pumped from the river.

The neatwork of all pedestals was poured in continuous runs, which necessitated working at night on a few of the largest pedestals. The maximum run of 215 cu. yds. for the pedestals of bent No. 24 was made in 24 hours, using one mixer. Most of the concrete work was carried on during the winter. The sand was heated by steam pipes to about 100° F., and the water was heated to about 150° F. The stone was not heated. After being poured the pedestals were housed in, and live steam was turned on for a period of about four days. The results produced were excellent, even during the coldest weather. The quantity of concrete in the pedestals and abutments is 11,850 cu. yds.

The concrete in the abutments and pedestals situated on the slopes at the east and west ends of the viaduct was mixed at the foot of the slopes, and was hoisted in concrete cars to the pedestals and abutments by means of an inclined track. The concrete in the pedestals located on the flat portion of the valley was placed with a stiff leg derrick.

The work connected with the layout of the viaduct was done by means of repeated horizontal measurements with a 100-ft. steel tunnel tape. A spring balance was used, and the tape was supported every 33 ft. Corrections were made for temperature changes. At each measurement point, an 8-in. diameter oak post, 5 ft. long, was set in the ground, and a zinc plate was tacked on top of it. This enabled the establishment of an accurate and permanent point. Independent lines were run to establish the 100-ft. stations, and the bents were checked from these points. On the slopes at the east and west ends, slope measurements were also taken as a check. Much of the measuring was done at night to secure the benefit of a more even temperature. The layouts provided for a normal temperature of 70° F. Concrete bench marks, extending 5 ft. into the ground, were placed so that direct readings could be made from some bench mark to every pedestal. These bench marks were checked independently by three sets of levels run at different times by different men, the greatest difference of elevation being 0.008 ft.

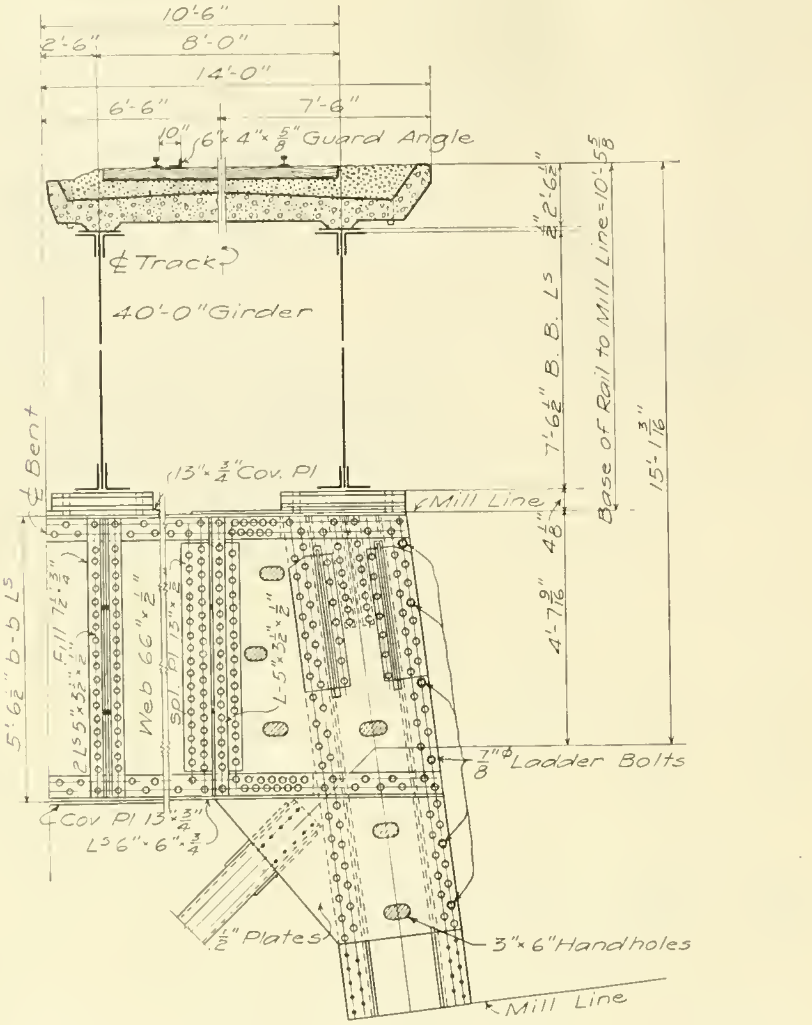


Fig. 4. Half Cross-Section Showing Floor and Details of Top of Bent—Des Moines River Viaduct.

The west bank of the river, for a distance of about 700 ft. above and about 300 ft. below the viaduct, was thoroughly riprapped, as the soil is soft and the bank had been rapidly wearing away for some time. Further protection was obtained by wing dams built up to the ordinary high-water level. Riprap was also placed around each of the pedestals of bents Nos. 3 to 24, inclusive, as a protection against scour during the flood period, when the water frequently covers the entire flat.

THE SUPERSTRUCTURE.

The deck girders are spaced 8 ft., center to center, and are carried directly on top of the cross girders (see Fig. 4). They differ slightly from the ordinary standard Chicago, Milwaukee & St. Paul Ry. type of deck girder in that the web does not project above the top flange angles; cover plates being used instead, in order that the concrete deck slabs should have central bearings on the girders. These slabs are provided with bearing strips, which rest on the cover plates between the inner lines of rivets. At the ends of the 70-ft. girders, where the section does not require the cover plate, a plate of the same thickness as the cover plate, with a width only sufficient to engage the rivets on the inner lines of the flange angles, was used to keep the bearing surface of the slabs at the same elevation. Holes for future staging are provided in the outstanding legs of the stiffener angles of all girders and cross girders. Over every bent one girder span has ladder bolts which extend through the outstanding legs of the two outside end stiffener angles, the bolt heads being countersunk in the end stiffener angles.

The cross girders are box girders, diaphragms being placed directly under the two inside deck girders, and also at the splices of the web plates and gusset plates. At the ends the webs are shop spliced to the gusset plates, which in turn are shop riveted to the outstanding legs of the column angles, the columns being spliced below the gusset plates (see Fig. 4). These gusset plates are also slotted on one side of the cross girder to permit the gussets for the longitudinal bracing to be reached and riveted to the column webs and angles. This construction is easily erected and presents an excellent appearance. Handholes in the gusset plates and ladder bolts at the ends of the cross girders provide easy access for inspection and painting.

The upper and the middle sections of the columns in all the bents, except Nos. 23 to 26, inclusive, are of the common built-up channel section, consisting of two 26x15/16-in. webs and the four 6x6x¾-in. angles. The rivet holes in the 15/16-in. webs were drilled from the solid, except such holes as were required in assembling. After the holes in the 6x6x¾-in. angles were sub-punched, the angles were bolted to the webs through the above mentioned assembling holes. The angles were then reamed with a twist drill which passed entirely through the webs.

Both the longitudinal bracing and transverse bracing are double systems, designed to resist both tension and compression. They consist of laced channels, with the flanges turned inward. One diagonal of the longitudinal bracing is in all cases carried through the intersection as a single member, while the other one is broken. This is done to provide an additional factor of safety against the sagging of these members. The tendency to sag is greatly increased by the fact that the longitudinal braces do not stand in a vertical plane, but are in the same plane as the columns of the towers. In the transverse system all the diagonals were broken at intersection points, the tendency to sag not being so great, as the bracing lies in a vertical plane and the method used made the erection somewhat simpler. The double system of bracing was chosen because of the increased rigidity—rigidity as well as ultimate strength being an important consideration in the design of high trestles. At the top the towers are braced in a horizontal plane by cross-frames, consisting of two 6x4x⅝-in. angles riveted back to back. These laterals are connected to large lateral plates which are riveted to the tops of the columns

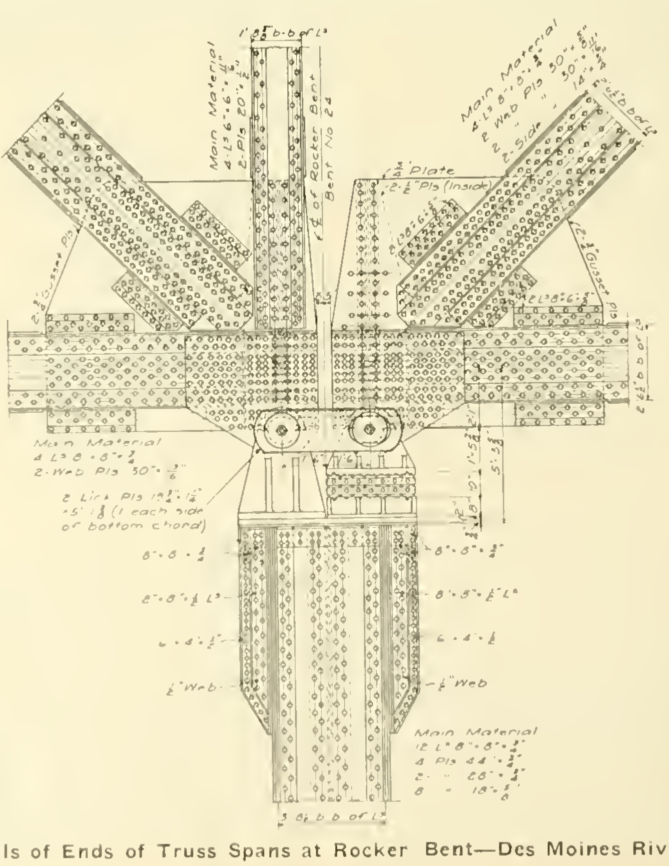


Fig. 6. Details of Ends of Truss Spans at Rocker Bent—Des Moines River Viaduct.

and cross girders. They are also riveted to the lower flanges of the inside tower girders.

Bents Nos. 22, 23, 25 and 26, which support the deck truss span (see Fig. 2, a), are similar to those above described, but are 1 ins. wider. The columns of the river bents (Nos. 23 and 24) are shorter than those for the adjacent bents as the shoes of the truss spans rest directly on the tops of these columns (see Fig. 5). A 2-in. plate is used under the bearings to secure a more equal distribution of the load over the column section. Struts are substituted for the cross girders in these bents. The longitudinal and transverse bracings in the shore towers are similar to the bracing already described, except that they are somewhat heavier. By referring to Fig. 5 it is seen that a single diagonal member is used at the top of each shore tower. This member is braced against sagging by an inclined transverse cross-frame. The horizontal thrust from the truss span is designed to be carried partly through the tower bracing by means of a horizontal strut, which connects the top of the river bent to a point at the same elevation on the other bent of the tower. In this manner the thrust from the truss span is equally distributed into all of the tower pedestals. The shoes on top of the river bents are each connected to the columns with eight bolts. Four of these bolts pass through the 2-in. base plate and connect to the horizontal cap angles on the longitudinal sides of the columns, two connect to angles on the transverse sides of the columns, the other two pass through the base plates and engage vertical lug angles which are riveted to the transverse sides of the columns.

On account of the heavy loads it was necessary to make the columns of the rocker bent (No. 24) much heavier than those in the towers. An H-section was decided on, both because of the simpler shop work and because of the advantage in having continuous webs in both directions. At the base each column rests on a cast-steel rocker-shoe, the connection being made with a 14-in. diameter pin. The load is further distributed by means of an I-beam grillage. The spaces between the I-beams were filled with concrete after the grillages were set and before the superstructure was fully erected. The main section of these columns is shown in Fig. 6, which also shows the manner in which the columns were widened in a longitudinal direction at the top to provide seats for the bed plates of the truss spans. The columns are reinforced with horizontal diaphragms spaced about 11 ft. apart. The flanges are braced by the transverse gusset plates and batten plates and by the single-angle lacing which consists of $3\frac{1}{2}\times3\frac{1}{2}\times7/16$-in. angles connected to each flange with four rivets. The gusset and batten plates are provided (as on the tower columns) with 3x6-in. hand holes. The transverse bracing in the rocker bent consists of four 8x6x½-in. angles, laced horizontally with $3\frac{1}{2}\times3\frac{1}{2}\times\frac{3}{8}$-in. angles and vertically with 3x7/16-in. bars.

As is shown in Fig. 6, the west truss has a fixed bearing on the rocker bent, while the east truss is provided with segmental rollers. With this arrangement the tractive forces in the truss spans are equally divided between the two river towers, whereas, had the expansion been placed at either of the river towers, the bracing in the opposite river tower

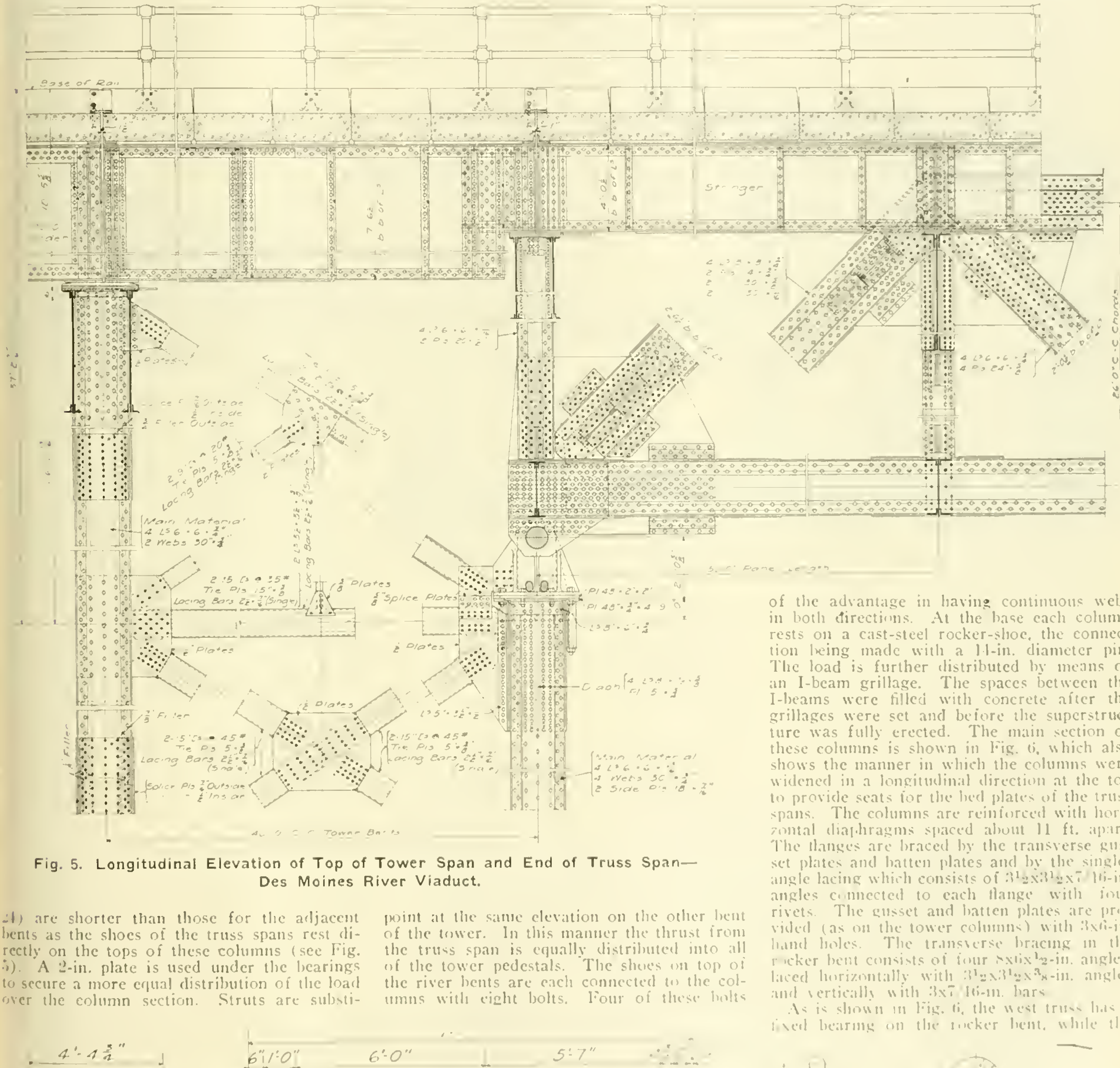


Fig. 5. Longitudinal Elevation of Top of Tower Span and End of Truss Span—Des Moines River Viaduct.

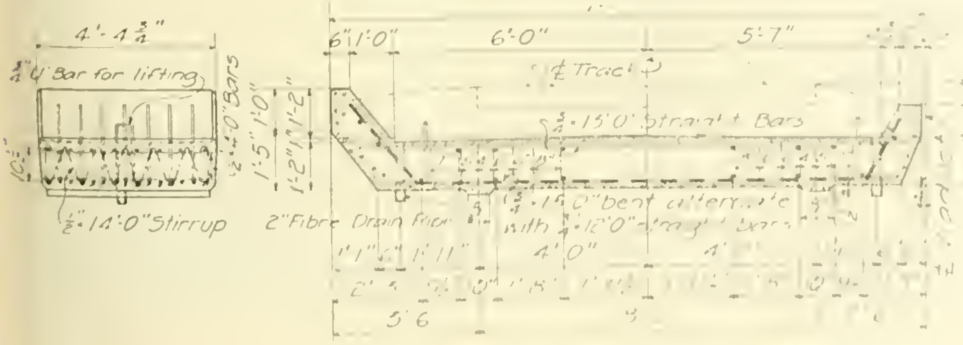


Fig. 7. Cross-Section of Reinforced Concrete Floor and End View of a Section of the Floor—Des Moines River Viaduct.

Fig. 8. Details of Hooks Used in Erection of Des Moines River Viaduct.

must necessarily have been very heavy. Large links, having one round hole for the fixed end and one slotted hole for the expansion end, engage the pins on each side of the shoes and limit the movement due to expansion and contraction. The links are provided in order that the spans will still be held at the rocker bent in case of the settlement of either shore tower.

The channel spans are double-track, riveted deck truss spans, each span having two trusses of the Warren type. The top of the floorbeams is flush with the bottom of the top chord, the stringers being placed on top of the floorbeams, their spacing being the same as for the girders on the remaining spans. This arrangement permits the use of a uniform floor from end to end of the viaduct. It also prevents the projecting ends of the floorbeams from marring the appearance of the structure.

In the end panel of the truss shown in Fig. 5, the stringers are riveted to the floorbeam which joins the vertical posts and is directly over the end bearing points. The 10-ft. girder spans, which also rest on this floorbeam, have provisions for expansion at their bearings on the floorbeams. This detail enables the vertical post to move when the truss span deflects without causing excessive bending stresses. The thrust frames, which are located in these panels in a plane below the stringers, transfer all the thrust to the truss at the intersection of the top chord and end post.

FLOOR SYSTEM.

The floor construction over the entire viaduct consists of a reinforced concrete slab which carries the ballasted track (see Fig. 7). The outer edges of the slabs are brought up to the elevation of the base of rail to give the deck a more substantial appearance. Refuge platforms, formed by two special slabs having the outside parapet extended, are provided about every 150 ft. These platforms are alternated throughout the length of the viaduct.

The concrete bearing strips of the slabs bring the center of bearing over the center of each girder, instead of beyond the vertical leg of flange angle, as is the case where the web projects above the top flange angles and the slabs are notched. These strips permit the top lateral plates to be riveted to the top flanges of the girders without countersinking the rivets, and also permit the lateral plates to be painted. The cover plate resists any tension which is brought into the flange of the girder from the lateral system. If the cover plate had not been used the laterals would cause tension on the rivet heads in the vertical leg of the flange angles. The tendency of the slab to move laterally, which is very small, is taken care of by the rivet heads along the inner gage line of the top flange.

Lift stirrups are provided in the slabs to facilitate handling. A 2-in. diameter fiber pipe was placed near each side and at the middle of each slab for drainage purposes. The slabs were all cast during cold weather. The forms were housed, and were heated by means of steam pipes underneath the floor. The roofs of the slab houses consisted of trap doors through which the slabs were lifted after they had set.

At all expansion joints the slabs are fitted to lug angles, which are riveted with the outstanding leg flush with the end of the girders or stringers, as the case may be. The spaces between the slabs at all such points are spanned with ½x10-in. bent cover plates, which are anchored to the slabs on one side with fox-bolts, ½-in. in diameter by 8 ins. long. By this arrangement the expansion joints are at all times kept open.

Open holes are provided in the outside parapet wall of alternate slabs, through which are bolted the castings which support the handrail stanchions. A 2-in. diameter pipe railing is used (see Fig. 5). This is primarily for the safe-guarding of the section men, but it also gives a feeling of security to passengers.

A 6x4x⅝-in. angle guard rail is provided near the inner rail of each track (see Fig. 4).

ERECTION FEATURES.

The erection of the viaduct was begun at the west end, as the long, high fill at the east end was not finished until after the completion of the steelwork. A temporary "shoo-fly" track (see Fig. 1) connected the main line with the west end of the bridge and provided a storage yard for materials.

All material, except the lower stories and the castings of the towers (hereinafter referred to) was erected from the track level with a 50-ton derrick car, equipped with an 80-ft. boom and a 50-HP. engine.

The procedure which proved to be the most satisfactory one was to erect each bent complete, then to set the girders and laterals, and lastly to place the longitudinal tower bracing. This procedure is illustrated by the line diagrams of Fig. 2(a), in which bent No. 27 is shown completely erected, the derrick car being shown in position to place a 40-ft. girder span. The longitudinal bracing, which is shown dotted in the upper stories of the towers, is yet to be placed. Temporary ties and rails were laid as the work advanced. The material to be erected was brought out as far as possible on flat cars on the track opposite the derrick car, and was then carried to position, suspended from the boom. The bracing and laterals were handled with runner lines, while the girders were handled with the load line and were grasped with girder hooks, a detail of which is shown in Fig. 8. These hooks were found to be very effective. The maximum stresses in the derrick were produced during the placing of the 18-ton cross girders, with the boom reaching across a 70-ft. span. Figure 9 shows the erection traveler reaching across a 70-ft. span and placing a tower post.

The following table shows the shipping weights of some heavy members handled.

Member.	Wt. lbs.
Cross girder for single bents	36,300
70-ft. girder span	36,500
Middle section of column rocker bent	72,700
Long bottom chord member	58,000

All riveting was done with pneumatic hammers, operated at an air pressure of about 105 lbs. The air compressor was located on the west bank of the river, and the power was supplied by a 40-HP. gasoline engine. Twenty-three field rivets per ton of metal were re-

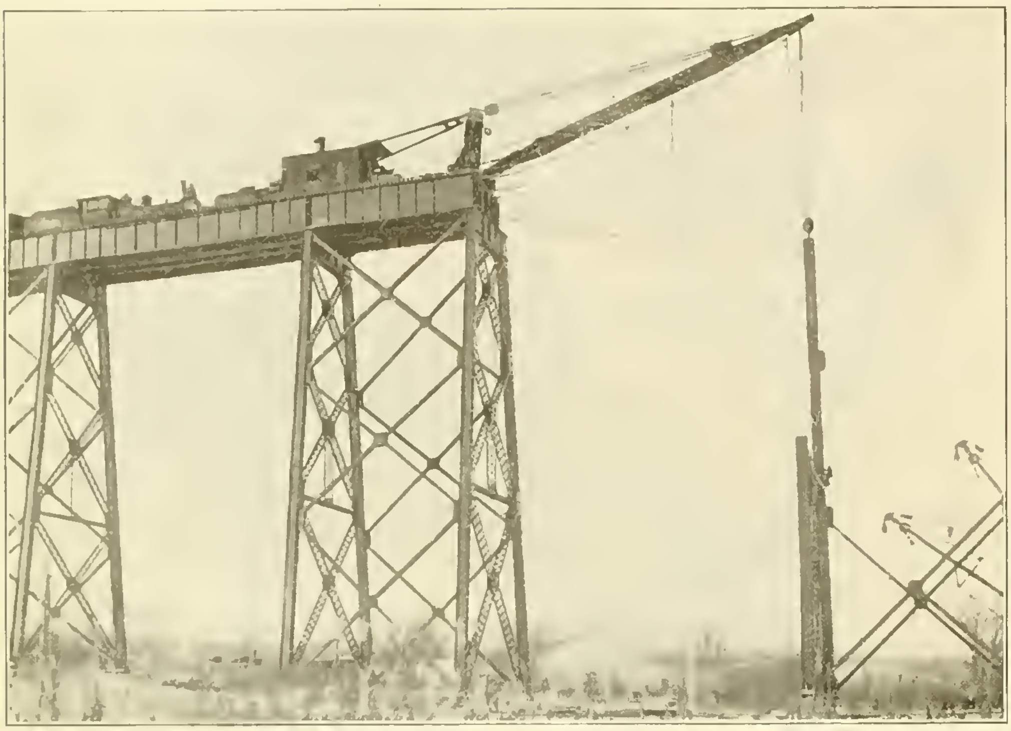


Fig. 9. Progress View Showing Erection of Towers of Des Moines River Viaduct—Boom of 50-Ton Derrick Reaching Across a 70-Ft. Span.

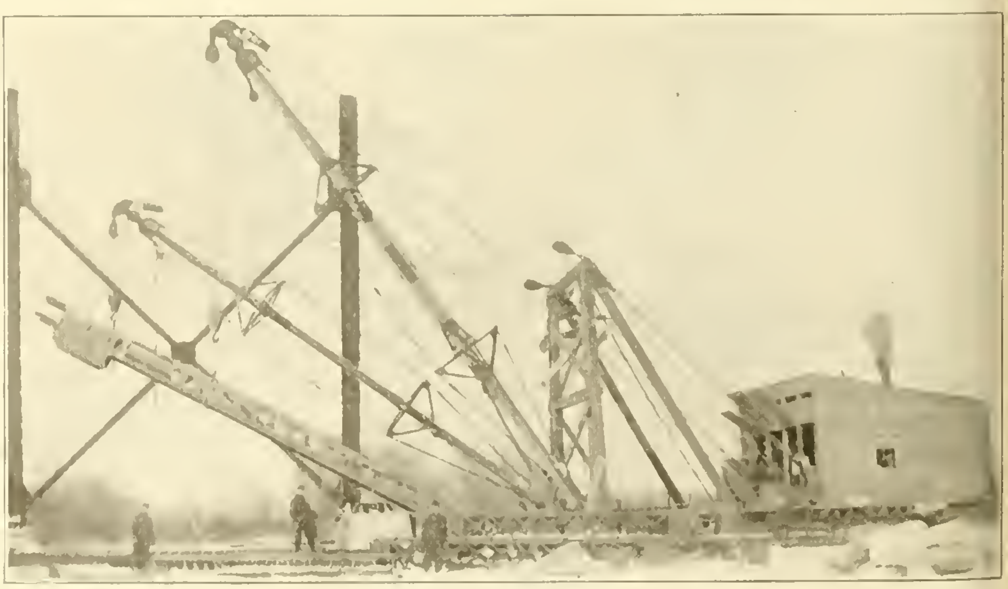


Fig. 10. Erection of Lower Stories of Towers With Double-Boom Mule Derrick—Des Moines River Viaduct.

quired for the trestle spans, while for the truss spans this number increased to 30. The riveting was carried on continuously from the time the erection started, the greatest number of rivets driven in one day being about 1,900, with four gangs working.

From bent No. 32 to bent No. 1 (see Fig. 2), where the comparatively level nature of the ground made it possible, the lower stories of the towers were erected by means of a mule traveler (see Fig. 10). This traveler was equipped with two 60-ft. booms and two double-cylinder engines, with double friction-drums and four independent winch heads. This traveler ran on an 18-ft. gauge track, laid along the center line of the viaduct. The material to be erected by the traveler was delivered on the "flat" east of the river, and was brought out for erection over a low-level temporary construction track, parallel to and 60 ft. south of the center line of the viaduct. It was originally intended to have the mule traveler assist in the erection of the upper stories of the towers by keeping the derrick car supplied with material. However, the shipments of the steelwork for the truss spans were so delayed that work on the lower stories was continued and entirely completed before the erection of the truss spans was begun. The "mule" was then moved out a sufficient distance to clear the pedestals, and was then run back to the truss spans to assist in the erection of the falsework. The procedure used in erecting the east portion of the viaduct with the mule traveler is shown in Figs. 2(b) and 10.

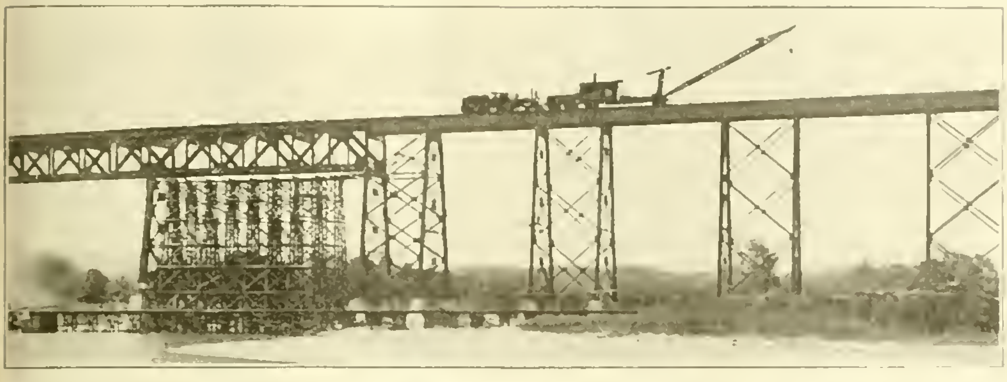


Fig. 11. View Showing Falsework Used in Erection of Truss Spans—Derrick Car on Completed Portion of Des Moines River Viaduct.

The truss spans were erected on the falsework trestle shown in Figs. 2(a) and 11, the trestle being supported on a pile foundation. The framed timber falsework bents, three stories high, were constructed of 12x12-in. posts, caps and sills. The longitudinal struts and all bracing were 3x12-in. timbers. The caps and sills were drift-bolted and the bracing was spiked. Each story of the falsework was framed in units. The units for the two lower stories of the bents were framed on the pile caps, and were lifted into place by the mule traveler. The units for the upper story were framed at the west end of the bridge and, together with the stringers and blocking, were placed in position by the derrick car as the work advanced. After the west truss had been placed the stringers and the units in the upper story of the falsework were taken down and again used under the east truss.

The most difficult problem in connection with the erection of the steelwork was the handling of the 72,700-lb. middle sections of the rocker bents. In order to do this work it was necessary to shorten the boom of the derrick car to 42 ft. The column sections were unloaded from the low level construction track (previously referred to), placed on carriages, and then run out on a trestle parallel to and 9 ft. west of the center line of the rocker bent. As nine strands of ¾-in. diameter wire cable were required for the load line of the derrick it was impossible to get sufficient line on the drum to reach members 115 ft. below the base of the rail. A 70-ft. loop, consisting of 10 strands of ⅝-in. diameter cable, was provided. This loop passed over a 50-ton hook on the end of the load line and through an erection casting at the bottom. The casting, when lashed to the column section, prevented the lines from cutting and at the same time permitted the member to be swung to the correct batter. To guard against any out-swing while lifting these heavy members the derrick-car track was temporarily curved outward, so that the end of the boom was nearly over the load when the boom was on the center line of the car.

Figure 12 shows a view of the west abutment and the west portion of the viaduct. This view also shows the construction track at the base of the viaduct and the derrick car at work on the east section of the viaduct.

The maximum tonnage erected in one day with both the derrick car and the mule traveler was approximately 200. It is to the credit of those in charge that not a single fatality nor serious accident occurred during the work. Ground was broken July 10, 1912, and the last span was completed July 12, 1913.

PERSONNEL.

The viaduct was designed by the engineering force of the Chicago, Milwaukee & St. Paul Ry., Mr. C. F. Loweth, chief engineer, and Mr. J. H. Prior, at that time engineer of design. The construction work was done by the railway company's forces, with Mr. [illegible] Howell in charge.

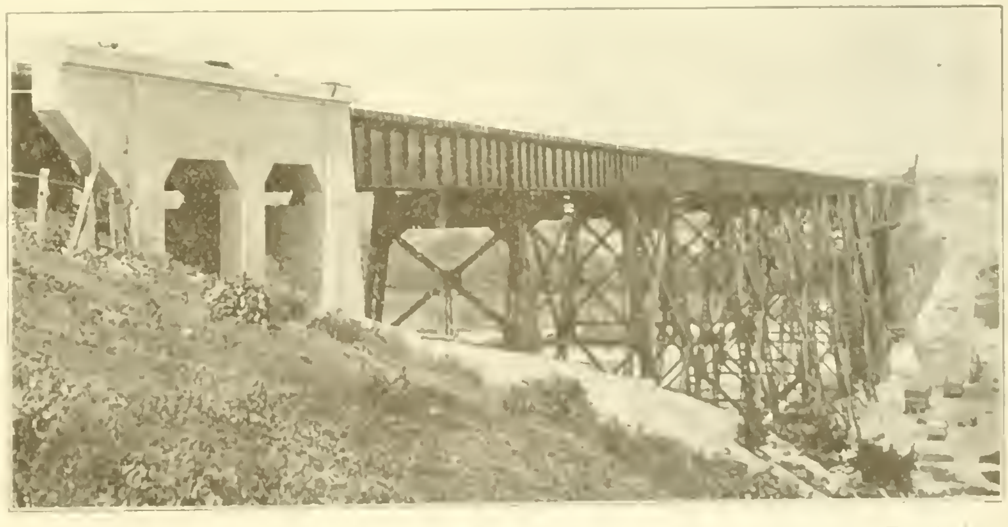


Fig. 12. View of West Portion of Des Moines River Viaduct—Construction Track is Shown at Base of Viaduct.

Exhibit of United States Steel Corporation at San Francisco.—The announcement is made that the United States Steel Corporation and its subsidiary companies propose to have a comprehensive exhibit of its operations at the Panama-Pacific Exposition in San Francisco in the year 1915. It will begin with the ore fields and carry on an educative picture of its operations in ore mining, rail and water transportation, coal operations, coal, coke and pig iron production, steel manufacture in all its various lines, and will also present in a materially displayed way the processes of manufacturing of many of its subsidiary companies' products, also the utilization of by-products and the display of many uses in which its general products are employed, typifying the advancement in the uses of this country's resources.

In addition to the material exhibits before mentioned, the corporation intends to exhibit in a comprehensive manner, by moving pictures, its operations throughout all departments showing the ramifications of the processes of the corporation's operations. It is proposed as well, to set forth the work which the United States Steel Corporation has done towards the social welfare of its employes and those depending upon them. Also it will exhibit many forms of safety devices that have been conceived by the corporation officials and its employes, and in the installation of which large sums have been, and are being, expended by the corporation.

Progress by Illinois Steel Co. in Developing Alloy Steels.—The Illinois Steel Co. has made considerable progress during the past year in developing electric steel, particularly nickel and chrome nickel alloys. It is now equipped to furnish electric special steels for any purpose. The greatest progress has been made in eliminating piping and segregation in the steel ingot. By adopting an inverted ingot and pouring with a refractory top the piping has been reduced to such an extent that the sale of electric steels on a competitive price basis with open-hearth steel seems not far distant. The most important improvement in manufacturing manganese steel rails consists of the installation of a quenching tank of large proportions. The toughness of manganese steel depends upon quenching from the temperature at which the rail is rolled, as contrasted with the air cooling of steel rails on the hot beds. To obtain the best results the bath must be of sufficient size and must have a circulation of sufficient activity to dissipate the heat from the rails without any appreciable rise in its own temperature. The quenching of the rails follows immediately after cambering, the rails being fed continuously into the tank. It has not yet been possible to overcome the obstacles which manganese steel presents to machining and the drilling of holes for bonding is still impossible. Punches have been designed and installed which give a satisfactory hole, but their operation is expensive, due to breakage of tools. The lack of machining qualities has operated against the rolling of manganese steel in shapes other than rails, although some manganese steel plates have been rolled. It has been possible to eliminate seams in alloy steel bars by chipping the seams in the billets. Experience has shown, however, that certain alloy steels are especially susceptible to stratifying when rolled from ingot form. Even a very slight reduction by rolling the billets through rolls does not give as good results as breaking them down by hammering.

Some Data on Several Large Railway Viaducts.

(Staff Article.)

In connection with the article on the design and construction features of the Des Moines River viaduct, published in this issue, it is interesting to note the type of construction, dimensions and weights of other large viaducts. We are indebted to Mr. J. H. Prior for the following data relative to the Des Moines River viaduct and five other important viaducts:

Des Moines River Viaduct.—The Des Moines River viaduct of the Chicago, Milwaukee & St. Paul Ry., near Madrid, Iowa, is a double-track structure having a length of 2,395 ft. 8 ins., from face to face of backwalls. The weight of the steelwork in this structure is 6,261 tons. The tower spans have a length of 40 ft., the intermediate spans a length of 70 ft., and the two river deck truss spans a length of 156 ft. 6 ins. each. The height of the viaduct, from river bed to base of rail, is 145 ft. An outline drawing showing its type of construction is shown in Fig. 1(a).

Boone Viaduct.—The Boone viaduct of the Chicago and Northwestern Ry. is located in Boone County, Iowa, about 17 miles north of the Des Moines River viaduct. It is a double-track structure having a total length of 2,685 ft., the weight of the steelwork being 6,196 tons. The tower spans have a length of 45 ft., the intermediate spans a length of 75 ft. and the deck truss river span a length of 300 ft. The height of the viaduct, from river bed to base of rail, is 185 ft. Figure 1(b) shows an outline drawing of this viaduct.

Kinzua Viaduct.—The Kinzua viaduct of the Erie R. R. spans the Alleghany River, near Kinzua, Pa. It is a single-track structure having a total length of 2,053 ft. 1 in., the weight of the steelwork being 3,350 tons. The tower spans have a length of 38 ft. 6 ins., the intermediate spans a length of 61 ft., and the two end spans a length of 62 ft. 0½ in. The height of this viaduct, from water level to base of rail is 301 ft. Its type of construction is indicated in Fig. 1(c).

Gokteik Viaduct.—The Gokteik viaduct of the Burmah Rys. Co., Ltd. (see Fig. 1, d) is at present a single-track structure, with provisions for a future double track. Its length, out to out of steelwork, is 2,260 ft., and the weight of the steelwork is 1,850 tons. The tower spans have a length of 40 ft., the intermediate girder spans a length of 60 ft., and the intermediate deck truss spans a length of 120 ft. The height of the viaduct, from its lowest point to the base of rail, is 320 ft.

Lethbridge Viaduct.—The Lethbridge viaduct of the Canadian Pacific Ry. at Lethbridge, Alberta, Can., is a single-track structure having a length of 5,327 ft., face to face of backwalls. The steelwork of this viaduct weighs 12,200 tons. The tower spans have a length of 67 ft. (see Fig. 1, e), the intermediate girder spans a length of 98 ft. 10 ins., and the deck truss span near the end a length of 167 ft. The height of the viaduct, from the river bed to the base of rail, is 314 ft., the tallest tower being 285 ft. high.

Viaduct over Kaiser Wilhelm Canal.—The viaduct over the Kaiser Wilhelm canal (see Fig. 1, f) is a double-track structure having a total length of 7,120 ft. Its height, from water level to top of rail, is 125 ft., and the weight of the steelwork contained in it is 11,300 tons.

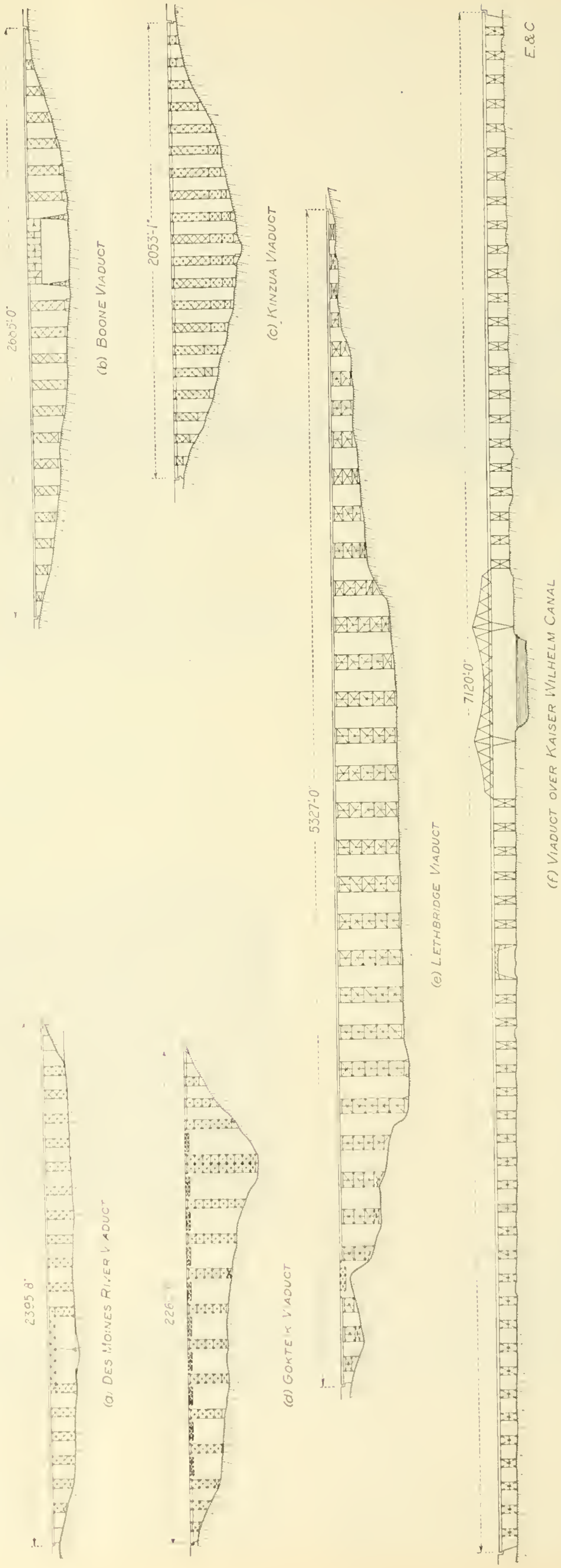


Fig. 1. Outline Drawings of Six Important Railway Viaducts Showing Types of Construction and Lengths.

Hardening Concrete Roads. The *Chemical Trade Journal* for April 18 contains the following:

A new hardening material for the surface of concrete floors contains 95 per cent of iron dust or iron flour. It is added to the dry cement in the proportion of 15 to 25 pounds to each 100 pounds and one part of the mixture is used with two parts of sand. This preparation is applied as a top coat to a thickness of ½ to 1 inch. It forms a hard and durable floor, claimed to be waterproof and not slippery, and is also used for making new concrete adhere to old in repairing.

Water Works

A Discussion on Equitable Hydrant Rentals and Better Methods for Apportioning Fire Protection Cost.

Probably no one thing has caused so much friction between private water companies and the municipalities served as the unscientific and still common method of charging for fire protection at a fixed sum per hydrant. In his paper before the 34th annual convention of the American Water Works Association, Mr. John W. Alvord of Chicago pointed out some of the evils resulting from unscientific apportionment of fire protection costs and showed how the existing evils can be remedied. The paper follows:

In the formation of so many new utility commissions as are now created, and are being created, there arises the opportunity to obtain a more equitable and scientific adjustment of rates than has hitherto prevailed. The Wisconsin commission has already done valuable work in showing that many of the municipalities in Wisconsin do not pay enough for public fire protection; that is to say, that the cost of the public fire protection system is in considerable part loaded onto the private consumer. It is a matter of common knowledge among those who study this subject that this is largely true all over the country. This condition doubtless arose from the fact that when franchises were originally granted, it was easier for city councils to grant and water company promoters to obtain concessions for a slightly larger domestic rate, which came from many small consumers, than a proper and just public payment coming from a usually insufficient and ordinarily depleted general fund.

The injustice of this proceeding is fully evident when a careful study of the two services is made. The valuable paper on this subject by Messrs. Kuichling, Hawley and Metcalf, presented to the American Water Works Association at the Rochester meeting (see ENGINEERING AND CONTRACTING of June 7, 1911), shows generally that with the smaller cities fire protection cost rises to a 65 or 75 per cent proportion of all the service, while in the larger cities this may be as low as to 20 and 25 per cent.

The Wisconsin commission has, as the result of its studies, generally raised the fire protection cost, and correspondingly lowered the domestic rate in the cases cited in Table I.

TABLE I.—CHARGES FOR FIRE SERVICE IN SEVEN WISCONSIN CITIES.

Date of Report.	City.	Population 1910.	Per Cent of Water Works Charged to Fire Service.	Annual Charge for Fire Service.
1910	Jefferson, Wis.	2,582	75.0	$2,560
1910	Ripon, Wis.	3,739	65.0	6,082
1911	Oconto, Wis.	5,629	57.5	9,812
1909	Ashland, Wis.	11,594	54.7	20,480
1911	Janesville, Wis.	13,894	54.0	15,795
1911	Beloit, Wis.	15,125	48.0	11,863
1910	Madison, Wis.	25,531	49.6	*17,000 About

*Special local conditions favor a low fire service charge at Madison.

It is safe to say that in most instances where private utilities are serving the community, the city is paying less than half of the amount which properly should be paid for this service, thus loading the balance of the cost usually onto the small private consumer.

In the case of a municipality which owns its own plant, and has largely paid for it from issues of bonds, paid from annual taxes, the fixed charges are perhaps more equitably distributed, but it is yet an open question, even in such cases, if a careful study would not disclose that the consumer is not paying more than his proportion for the public service, by reason of the fact that he virtually bears all the operating cost.

METHOD OF EVALUATING FIRE PROTECTION SERVICE

The proper method of dividing the cost of public and private services is, (1) to make a valuation of the plant; (2) to make a study of the value of a theoretical plant, sufficient in capacity for fire service only, and (3) a study of the value of a theoretical plant, capable of domestic service only. The sum of the (2) and (3) study will give a value much greater than the value of the combined plant, and the value of the combined plant should then be divided between fire service and domestic service in proportion to the theoretical values found necessary in each case. To illustrate: If the cost of a combined fire and domestic service in a single plant is found to be $100,000 for a given community, and the cost of a strictly fire service plant is found to be $80,000, and a strictly domestic supply plant $70,000, or a total for two separate plants of $150,000 for the same community, then the value of the combined plant should be divided between the two services on the basis of eight-fifteenths and seven-fifteenths of the total value of the combined plant, or about $53,400 for the proportionate investment for fire service, and about $46,600 for the proportionate investment in domestic service.

In addition to this study, it is necessary to inquire, in the same manner, into the operating expenses, and divide the combined operating expenses between those chargeable to fire service and domestic service. Here it will be usually found that domestic service carries the higher charge.

Having found the total investment in each service, and computed its interest and depreciation, and added to this the proper proportion of operating expense, we have a proper and equitable division of annual revenue which should be derived as between the private consumer, on the one hand, and the public fire protection expense, on the other, and, as has been before said, it will be usually found that the public is not paying nearly enough for the public service and the private consumer is bearing the burden.

Here, therefore, is an opportunity for our public service commissions to do some scientific rate making, easing the burden particularly on the small consumer and placing it where it justly and properly belongs, on the general public.

But more than this can be done. When the proper amount for the public to pay for general fire protection is properly ascertained, it can be more scientifically applied than under the present irrational method so that it will not result in the constant friction that the present method involves.

The method of paying for fire protection at so much per hydrant is very common, but it is illogical and improper for the following reasons:

1. It does not apply the payments to what the public really pays for, viz., pumping capacity, distribution capacity, and the size and number of fixture openings.

2. The present method has a tendency to contract the fire fighting efficiency, by concentrating the payment on an outlet fixture, thus constantly tempting the cities to improper economy by the neglect to install the proper number and closer spacing of the hydrants, with increasing fire risk in any given district.

3. The present method spreads the cost of fire protection inequitably, in that cities have to pay for outlying districts with low fire protection risks, as much per hydrant as in central business property, with high protection risks.

4. It discourages the extension of the distribution system into outlying districts, by reason of the relatively high cost of the hydrants needed.

5. It does not encourage the use of adequate sized mains, because attention is concentrated on the outlet fixture and its spacing rather than the size and capacity of the distribution system.

6. It discourages the setting of additional and intermediate hydrants when the adjacent fire risks increase, even though capacity of mains may be available.

7. It makes for constant contention between the city and the company whenever new hydrants or new extensions are needed, by reason of the above facts, which neither the company or the city officials are always able to fully analyze or understand. Thus we have these two important interests in constant antagonism where both should be in harmony in working for a common end at a just cost.

Many of the bitterest contests between private companies and municipalities arise from this cause.

Now, all this source of contention and friction can be entirely avoided by a more proper method of payment for the public service, which will eliminate the objections above made, and place the burden of cost where it belongs; that it is to say, in the capacity of the system for fire protection rather than on the number and spacing of outlet fixtures. Just as it is more equitable to sell water to the private consumer per unit of volume, by meter, rather than by number of outlets in the consumer's home, so is it more equitable to pay for fire protection per unit of capacity for fire protection purposes rather than mainly by unit of outlet fixture. In other words, let us pay the company for the amount of capacity it has in its pumping and distribution system, as well as for the number of outlets and we will have destroyed most of the sources of friction that now exist.

The method that first suggested itself to the writer was to apportion fire protection cost in part to pumping capacity, in part to pipe distribution capacity, and in part to outlet, but on reflection it was seen that this triple divison produced some complexity, which was not really needed, and which did not make for simplicity.

It is comparatively easy for cities to enforce the installation of adequate pumping capacity, and where pumping capacity does not amply exist it is not difficult for water companies to enlarge it in fairly reasonable length of time. Attention is constantly directed to deficient pumping capacity and its future lack is not difficult to foresee.

But the enlargement of distribution pipe capacity up to the level of a constantly growing population is not so easily accomplished. Its lack is produced by insidious neglect of which the manager, the administration, and the public has no warning. It is here, if anywhere, that we need in the ordinary plant and city to have an incentive to upkeep and enlargement. It would, therefore, appear that for purposes of simplicity, it would be fully as effective to proportion fire protection payments in large part on pipe distribution capacity, and in small part on outlet fixture payments, as to proceed in a more complex manner by including pumping capacity.

With this principle in view, and having arrived at the proper amount to be paid the utility for fire protection by the methods herein first described, it remains to apportion this amount in such a way that incentive is introduced to properly install mains and distribution system of the proper size, and hydrants at the proper spacing so that the fire risks will be proportionately met in different sections of the town, and at the same time, so that extensions and additional hydrant rental will not be burdensome, and, in short, that all or most of the causes of friction between the municipality and the utility enumerated above will be removed.

This can be done by paying for the fire service in large part per mile of pipe at rates depending on its capacity.

For the purpose of having some regulating limit, it is also desirable to pay for the hydrants a small sum, which represents a reasonable return on their cost of installation, together with an allowance for their depreciation and annual maintenance. This in reality amounts to a nominal sum, generally not more than $5 to $8 per annum per hydrant. At this rate it should be the privilege of the city to order in as many or as few hydrants as proper attention to the relative fire risks of the different sections of the city would seem to demand. A proper appreciation of the rapidity with which long lines of fire hose reduces pressure would undoubtedly make for a liberal allowance of hydrants under such a scale of payment.

The company, being paid an annual sum for the hydrants and their maintenance, has no separation of interest from the viewpoint of the municipality.

The bulk of the payment for fire protection under such a system should come from the pipe capacity unit.

The pipe capacity unit does not need to be scientifically exact; it is sufficient for all practical purposes if it is only approximate, and particularly desirable that it be simple.

The best unit which has occurred to the writer, that approximates in some degree the relative cost and capacity of mains sufficiently for this purpose, and is at the same time simple, is the inch foot of diameter; that is, the number of inches of diameter 1 ft. long in a given system of distribution pipe. Thus, a 6-in. pipe is rated at 6 inch feet to every foot, and a 12-in. pipe 12 inch feet to every foot.

Having found the total amount of money to be paid for fire service in a given system, we should deduct the amount that will be raised by the nominal hydrant rental before described, and we should also deduct the amount which will be raised from sprinkler systems and other especial methods of fire protection, and the remainder will be the amount to be paid by the city on the pipe capacity unit or inch foot basis. The total number of inch feet in a given system, divided into the amount to be raised by this part of the protection, will give us the cost per inch foot. This will probably fall in most normal cases within .4 ct. to .7 ct. per inch foot.

The total annual payment then for any given pipe will be its diameter times the inch foot unit times its length.

Thus, if the inch foot unit is a half a cent, a 6-in. pipe will have a fire protection rental of 3 cts. per foot per annum, and a 12-in. pipe will have a rental value similarly of 6 cts. a foot annually.

To this amount computed for all the pipe lines should be added the nominal hydrant rental computed on the total number of hydrants, the total making the annual public fire protection bill to the city.

It is true that the cost and capacity of mains increase at a rate greater than the first power of the diameter. The capacity increases as the square of the diameter, and the cost between the first power and the square. If the cost of mains is increased by the replacement of cost cutting through payments, the adjustment of cost as between different diameters will be closer.

A uniform rate per inch of diameter is a slight inducement toward the smaller pipe, as that has the larger return, and this may be equalized (if it is thought wise) by a sliding scale, but, as a matter of practical effect, it is not believed that the variation will have any appreciable influence in keeping down the capacity of the distribution system.

The advantageous features of this arrangement are:

1. That the city proportions its payment to the amount of distribution pipe capacity it receives, as well as the number of fixture openings (hydrants) that are available.

2. The payments for fire protection are usually and normally in proportion to the fire risks in the different districts, because the larger mains are in a general way either within or lead to those districts most fully built up and most needing fire protection.

3. There is every incentive to the company in extending its distribution system, to keep sizes of mains up to fire requirements, and to reinforce weak districts, and the cities' interest in fire protection and the companies' interest in proper remuneration are thus coordinate.

4. In outlying districts the city does not have to pay for protection out of all proportion to the capacity needed, and it may further regulate the number of hydrants in such districts to suit their character and growth.

5. There is no obstacle to the introduction of new hydrants when given districts need increased fire protection, and thus the fire department is rendered much more efficient.

The writer, in company with Dean F. E. Turneaure of Madison, Wis., and Dean Marston of Ames, Iowa, was appointed in 1912 as a board of arbitration to determine the value of the water works property at Freeport, Ill., decide upon the necessary enlargement of the plant, fix a schedule of fair rates, and draw up an ordinance for the extension of the franchise, should the city determine to extend rather than purchase. (For abstract from this report see ENGINEERING AND CONTRACTING of Oct. 2 and Oct. 9 and Dec. 4, 1912.)

This opportunity was taken to introduce the method of paying for fire protection described herein, and the city electing to extend the franchise, the method of paying for fire protection has now been in force there for the last two years, and has given satisfaction.

The value of the Freeport plant in April, 1912, was found to be $316,000 (population about 19,000). New extensions and enlargements were recommended which would increase this amount to $502,927. A fair return on this amount was recommended to be such that it would at no time fall below 7 per cent, after due allowance for all expense of operation and depreciation. This called for a return within four years of at least $70,000.

From a careful analysis, it was found in this particular case that 44 per cent of the total investment was necessary for fire protection, and 56 per cent for domestic consumption, and it was recommended that the city raise its payment for fire protection from a total of $10,000 (the amount then paid) to $15,000 in 1915, and $19,800 later. A corresponding reduction was made in the rates to small consumers.

The apportionment of the fire protection cost was made on the basis herein described, and resulted as follows:

For each hydrant the annual sum of $7.50 to cover interest, maintenance, and depreciation.

For future extensions as follows:

$0.10 annually per foot of 20-in. pipe.
.08 annually per foot of 16-in. pipe.
.06 annually per foot of 12-in. pipe.
.05 annually per foot of 10-in. pipe.
.04 annually per foot of 8-in. pipe.
.03 annually per foot of 6-in. pipe.
.02 annually per foot of 4-in. pipe.

No pipe smaller than 4 in. was recommended, and 1 in. not to be used for greater lengths than 1,000 ft. between cross mains.

For private fire protection services:

Private hydrants $7.50 annually per hydrant.

$0.06 annually per foot of 6-in. supply pipe.
$0.04 annually per foot of 4-in. supply pipe.

The charges per foot of private fire protection supply pipe being higher than charged the city, because such supply pipe affords no revenue from domestic consumers.

For sprinkler service:

An initial payment of the entire cost of the supply pipe, safety valves, etc.

For each sprinkler head $0.05 annually.

Minimum charge $50 per annum.

The above findings are only given to illustrate the working out of the system. Each case requires to be investigated by itself, and may result in very different figures owing to special local causes.

It is to be hoped that the suggestions here contained will commend themselves to those cities or companies about to renew fire protection contracts, so that past causes for fruitless difficulties and friction may be avoided, and especially is it possible for the new public utility commissions to introduce such a method as is here outlined, because they have it easily within their power to adjust rates to a just and scientific basis, equitable to both the public and the private consumer, without bargaining or pressure from political or personal interest.

Results of Experiments Made to Determine Rate, Manner and Factors Influencing Death of Bacteria in Drinking Water and Polluted Streams.

Data on the rate and manner of the death of bacteria in drinking water and in polluted streams, and the different factors which influence this death, are of considerable importance to water works men since, in the case of pathogenic bacteria, these matters affect the health of men and animals. A knowledge of the longevity of B. coli is of aid in determining the length of time a sample may be kept between sampling and analysis and it may also be of value in determining the volumes to be used in the analysis of the sample. A knowledge of the longevity of B. typhosus is necessary for the purpose of analysis for the organism itself, and also to determine the chance of infection from a stream which may be contaminated with that organism. The results of experiments made to determine the rate, manner and factors influencing the death of bacteria in drinking water and polluted streams were given in their recent paper before the Illinois Water Supply Association by Dr. Otto Rahn, assistant professor of bacteriology, University of Illinois, and Mr. M. E. Hinds, assistant chemist of the Illinois State Water Survey. Their paper follows:

As a number of uncertain factors would be involved in working with a natural water or sewage, it was thought best to work first on pure water under known conditions and to vary the conditions until they finally approached those found in natural waters. The water used for this work was ammonia free, which, however, contained a very small amount of nitrogen. It is purer than ordinary distilled water, being redistilled twice, and is the purest grade of water obtainable unless a large amount of labor is spent on its preparation.

TABLE I. — THEORETICAL NUMBER OF CELLS PRESENT AT END OF 60 HOURS AND FOR EACH 6-HOUR INTERVAL.

Hours.	Theoretical number of cells.	Actual number of cells.
0	1,000,000	315,000
6	100,000	40,000
12	10,000	937
18	1,000	
24	100	
30	10	
36	1	
42	.1	
48	.01	
54	.001	
60	.0001	

The rate of death of B. typhosus as found by different observers is variable, probably due to different experimental conditions. Of the many contradictory statements concerning the death rate, it is difficult to determine which one is correct as most of the data appear to be reliable. This leaves us in doubt as to whether death is due to lack of food, presence or absence of oxygen, temperature changes or antagonism of other bacteria.

During the last seven years we have learned that bacteria under unfavorable conditions die gradually. Of the total number, a certain percentage will die in a unit of time and of the number surviving the same percentage will die in the next unit of time. As far as we know, this law holds true with all causes of death, whether by disinfectants, light, drying or heat. We know of no exception.

It was only natural to expect this same regularity of death to hold true in the case of B. typhosus and B. coli in pure water. The experiments showed this to be the case. The reduction in the case of B. typhosus was 89.3 per cent in six hours, that is, after six hours' time only about 10 per cent of the original number survived. Assuming 1,000,000 cells to be present in the beginning, we find the numbers remaining as shown in Table I, which is based on a reduction of 90 per cent in each six-hour period.

It is easily seen that we never come to an absolute zero. There are always some bacteria left alive, but the number soon becomes so small, that for practical purposes, we might consider them absent. Table I shows a reduction in 60 hours from 1,000,000 per c.c. to less than 1 per gallon. Whether such water would be considered safe for use, is questionable. In 96 hours, we would have less than one typhoid bacterium per 1,000,000 gals. Such water would probably be safe. Certainly, no bacteriological or other analysis could discover the bacterium.

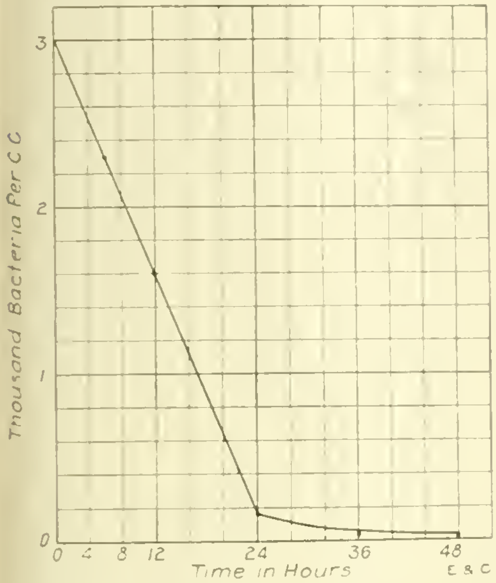


Fig. 1. Diagram Showing Death Rate of B. Typhosus in Pure Water.

B. coli does not die quite as fast as B. typhosus, about 72 hours being necessary to reduce their number from 1,000,000 per c.c. to 1 per gal. Kruse states that B. coli is found a normal inhabitant of all waters, whether good or bad. We can find it, if we only take a large enough sample.

Knowing the facts, we tried to find the cause of death. It is probable that the death of B. coli and B. typhosus in pure water is due to starvation. A sample of tap water was sterilized and inoculated with B. coli and proved to be a fair medium for growth. An initial number of 1,500,000 increased slowly to 3,000,000 in twelve days and then slowly decreased, over half the original number being still present at the end of five weeks, when the experiment was discontinued. At the same time a sample of deep well water with a very high mineral residue was sterilized and inoculated with about 2,000,000 B. coli per c.c. and only 1 per c.c. was found at the end of two days. This death rate was higher than in very pure water. Only a trace of organic nitrogen was present.

It is very important from a practical viewpoint to decide whether or not dissolved oxygen plays any part in the rate of the death of bacteria in water. Whipple and Mayer found that B. typhosus died about 20 times as fast without oxygen as with it, and B. coli died about twice as fast. The absence of oxygen was secured by keeping the tubes in an atmosphere of nitrogen and hydrogen. This would suggest a suffocation of bacteria. In our work we have been unable to get the same results with B. coli as in all of our tests the death rate was lower without oxygen. Our work on B. typhosus bears out the previous work in that the death rate is higher in nitrogen than in air, but the difference is slight as compared with Whipple's, perhaps due to a different strain, perhaps to different water. Figure 2 shows the difference in death rate in air, hydrogen and nitrogen.

The temperature of the water has a direct bearing on the death rate. Even though B. coli and B. typhosus both grow much better at 37° C. than at 20° C., when cultivated under growing conditions, they also die faster at the higher temperature. With B. coli we have found the rate of death to increase 1.8 times with each 10° rise in temperature. Growth and death of bacteria must be looked upon as chemical reactions, and, therefore, must proceed faster at higher temperature within certain limits. In work with B. typhosus in Lake Michigan water, Russell found the death rate increased eight times from freezing to 12° C.

In 1911 Ruediger determined the rate of death of B. coli and B. typhosus in river water under natural conditions both in summer and in winter. He attributes the faster death rate in summer to the effect of light and to saprophitic plants. We can account for the change in death rate as a change due to the difference in temperature.

CONCLUSIONS

In pure, natural water and in re-distilled water, B. coli and B. typhosus die from starvation in the gradual, regular manner observed with other causes of death.

The rate of death increases with the temperature.

The presence of oxygen under these conditions, seems to be harmful for B. coli, but beneficial to B. typhosus.

Further work is to be done on the effect of organic matter, mineral matter, other organisms and the temperature coefficient under these varying conditions.

The Use and Benefits of Pressure Recording Gages in Water Works Operation.

Recording gages for keeping a permanent record of steam and water pressures and vacuum have been used in water works plants for many years. As long ago as 1889 Mr Edwin Darling said: "No well conducted water works can afford to be without recording gages and, when properly located, they will, in my opinion, pay for themselves within one year." By reference to the gage charts the pressure which existed at any time, day or night, and on any date can be determined. The recording gage is a constant watchman just as a water meter is an ever-present inspector. Despite the fact that such gages have so long been appreciated in the more efficiently managed works they are not used in many plants and often, where installed, they are not located or utilized to the best advantage. The present discussion on the use and benefits of pressure recording gages is taken from the paper by J. M. Diven before the 34th annual convention of the American Water Works Association.

USE OF GAGES IN PUMPING STATIONS

A recording gage on the steam line is the best possible check on the work of the fireman and engineer. If pumping to a reservoir under a constant head, the uniformity of the pressure line tells how uniformly the fires are being tended and the feed water applied. An absolutely even line would indicate perfection in the fire room, a ragged line, inattention on the part of the fireman. A sudden drop in the pressure line should correspond exactly with the record of fires cleaned or boilers blown off. A quick drop or sudden rise at any other time indicates improper firing.

With a direct pumping system, or where pressure has to be raised for fires, a less even steam line is to be expected, but a comparison of the steam and water charts will show how quickly the boiler responds to a sudden demand for more water pressure, by restoring and holding the pressure when the work of the pumping engine is suddenly increased. A direct pressure pumping system requires greater care and alertness on the part of the engineer and fireman, and the recording gages are the best means of indicating how well they perform their work. Perfection would mean a line equal to one drawn with a compass or straight edge, according to the style of chart used. A ragged, zigzag line means inattention and carelessness, or something wrong with the boilers, steam lines, engines or pumps. It rests with the chief engineer or superintendent to find out which is at fault, and to correct the fault.

The presence of recording gages is an incentive and stimulus to the men to do better work. They know that they are constantly watched; that a constant and indisputable record is being made of their work. The superintendent, when he visits the pumping station, or when the gage chart is placed on his desk, has a perfect check on the work of the pumping plant force, and a record he can preserve. When he has occasion to call an engineer or fireman to account for laxness he has an undeniable record of work performed. A poor pressure line may not be the fault of the engineer or fireman; it may be due to poor fuel, a leaky or sagging steam line, bad feed water, poor draft, or engine trouble; but the engineer is just as much at fault, for not promptly recording and reporting such troubles, as he would be for neglecting his work, or not keeping the fireman up to his part of it.

If good pressure charts are obtained for a while, but constantly and uniformly get bad, it indicates either that the plant is wearing out or being overloaded. If both good steam and water charts are obtained when the plant is pumping say 5,000,000 gals. a day, and grow more and more ragged as the pumpage is increased, they surely indicate that the pumps or boilers, or both, are being worked beyond their economic capacity. If the pressure lines grow in irregularity, when there is no increase in the work, they indicate that some part of the plant is wearing out or is in bad order, and needs overhauling.

Steady lines of the steam charts indicate uniform and careful firing, and a steam plant in good condition. Ragged gage lines indicate either poor and irregular firing or bad condition of the boilers. If the feed water is a scale forming water, the poor gage lines bear out the evidence of the coal scales, that the boilers need cleaning.

A fireman to maintain an even line on his steam gage charts must stay by his fires, fire frequently and lightly and keep clean fires. Knowing how completely he is on record he will be careful and will also keep a record of everything that might affect his record, such as poor fuel, time of cleaning fires, blowing of boilers, etc. The latter records, which might otherwise be neglected, are in themselves of sufficient value to more than pay the cost of the recording gages, if no other benefits were obtained.

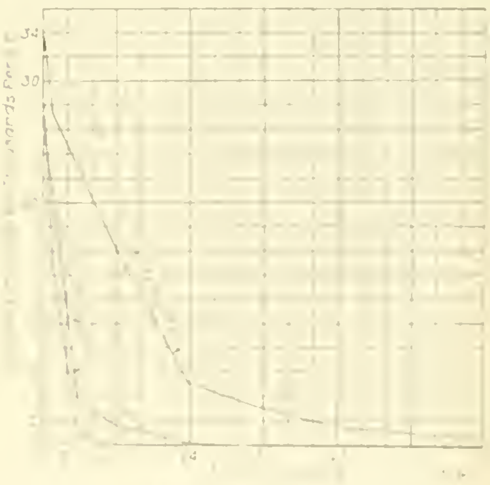


Fig. 2. Comparative Death Rates of B. Coli in Water With Air, Nitrogen and Hydrogen Atmospheres.

The vacuum gages should also be recording, to indicate the condition of the engines and pumps, a falling off in the height of the vac uum line indicating leaks that mean more coal. The lines on the vacuum gage charts should not vary much. If a vacuum of 27 can be obtained one month or year, it should be maintained the next month and year. If a higher vacuum is maintained with one kind of packing than with others, the one holding the high vacuum is the best, and probably the cheapest, though much higher priced than the others. The difference in cost might be made up many times in the saving of fuel.

No engine room is complete without at least three recording gages, as the operation of the plant cannot be properly checked and governed without them. They also insure better work on the part of the operatives.

Many times the recording gages create a spirt of rivalry between men on different shifts, through an effort on the part of each shift to do better than the others, to show more even lines on the charts; thus causing all to do better and more economical work.

There should be a certain economical relation between the steam pressure and the water pressure. When this is worked out it should be as closely maintained as possible. An unnecessarily high steam pressure, even though it gives a very uniform gage line, is not economical.

USE OF RECORDING GAGES ON THE DISTRIBUTION SYSTEM.

On gravity supplies or pumping plants, in cities of considerable size, a gage, or gages, should be placed on the distribution mains at some central point, or at typical points. If more than one service, high and low service, each should have a recording gage. It is well, when practicable, to take gage services off of large or leading mains, where they will not be liable to the great fluctuations in pressure frequent on smaller distribution pipes.

Of course there should be a gage in the office, where it will be under the eyes of the superintendent when in the office, and, at other times, of some other officer or employe of the water works, of the night watchman, if one is employed; one in the superintendent's house, for reference at night, is convenient, but, with telephone connections, not so necessary.

The connection to the gage should be entirely independent of all other service to the building in which it is located. It should be of good size, not less than ¾ in., and placed where it will be in no danger of freezing, as there cannot be a circulation in the gage service and an accurate record of pressure on the main at the same time, without the expense of a double service, in which a free circulation would be doubtful, unless it were very carefully planned, and even then there would be an item of friction in the smaller pipe, that would be difficult to account for. Of course water cannot be allowed to run in the gage supply pipe, in cold weather, without totally destroying the value of the gage record.

The gage service should be so arranged as to be easily blown out or flushed, to avoid all possibility of stoppage, by an accumulation of sediment. While a ¾-in. or larger service is recommended, the gage operates with a very small opening; the object of the large service is to avoid stoppage.

LOCATION OF GAGES.

The pumping station gage comes first, and, in smaller plants, is all that is required. Larger plants should also have an office recording gage. Cities or towns with two or more services should have a record on each service. Large cities should have recording gages at various points on the distribution system, notably near the congested value districts.

Gages can be placed in the residences of employes of the works. They require little attention, winding of clock, changing of charts, and seeing that the pen, where ink records are made is kept properly filled.

The fire department stations are convenient places for recording gages, and the fireman's interest in the water pressure is enough to insure proper attention. For municipal plants, police stations can be utilized, if they are better located than the fire engine houses, that is, on larger supply mains.

Very long services should, naturally, be avoided, both on account of the expense and the danger of stoppage in long lines of small pipes. The service should be of durable material, and, with corrosive waters, iron services should be avoided, as an accumulation of iron rust would stop up the minute orifice of the gage.

OTHER BENEFITS OF RECORDING GAGES.

The benefits of recording gages are many; they have been known to stop law suits, where actions for damages caused by low pressure were threatened. The recording gage settles many disputes concerning pressure at time of fires or other times. Often complaints of low pressure are made, not enough to supply the upper floors of some buildings. A nearby recording gage demonstrates that it is a local trouble, within the building. These complaints are frequently that the pressure is low at certain hours of the day or night, but a gage chart showing a uniform pressure at those hours demonstrates that it is local use of water, in the building itself, or in the distribution system near it, that causes the trouble.

The recording gage charts show the effect of cold weather on the water supply, the low pressure lines on the chart in freezing weather indicate the extent to which water is being run to prevent freezing. They also reflect the hot weather use of water; a low pressure line on the chart at night tells of water allowed to run on lawns all night.

PORTABLE RECORDING GAGES.

All parts of a city cannot at all times be covered by recording gages, so that the story they have to tell is incomplete. Portable recording gages set up in various places for short periods make a fairly complete record of the conditions of the supply at points on the distribution not covered by regularly established recording gages. Such records in large buildings, factories, etc., where the fire hazard is great, are particularly useful.

A gage set up for a single day gives a valuable record, as it shows a comparison between the day and night pressure at some particular point.

Portable gages can, with advantage, be connected to fire services, as, on such services, they would be free from the fluctuations due to the use of water on the premises, or, if the fluctuations existed, they would reveal improper use of water from the fire services. With a small portable house or box to protect them, gages can, in warm weather, be attached to fire hydrants, to make a record of the pressures on the mains at any point. Summer records of this kind, made on the outlying mains, mains in sparsely built up parts of the city, on long runs of small distribution pipes, and on the outskirts of the town, are of great value and interest.

PRESERVATION OF GAGE CHARTS.

Charts should be permanently kept as a part of the records of the works; as part of the history of the plant. They should be conveniently filed for reference at any time. For straight line charts suitable albums or scrap books make a convenient file. For round charts the scrap book form, though suitable and convenient, is bulky, as a page would be required for each day's chart. Substantial pasteboard boxes of the right size to hold a year's charts are convenient. These can be labeled with the year, and, where several recording gages are in use, with the location of the gage, making it convenient to refer to the charts from any gage at any time.

It is interesting to look over the old gage charts, also often instructive. A new water works, with new and clean pipes, with pumps not overcrowded, and ample size force and distribution mains should show a steady line, with slight fluctuations, and near the outer edge of the chart. As the consumption increases, the line grows less and less steady, and converges toward the center of the chart, showing the reduced pressure due to greater friction in the mains. The raggedness of the pressure line on the chart also increases with the age of the plant, as the mains become tuberculated and clogged with sediment. Finally, the consumption has gone beyond the pump capacity; a new and larger pump is installed, and the pressure line again approaches the outer edge of the chart, and becomes more steady, but still shows greater fluctuation than when the plant was new and working at a comfortable rate.

Notwithstanding the new pumping engine, the results are not entirely satisfactory, when compared with the earlier charts; the pumping engine, though of ample capacity, is working harder than the original pump had to work; both the water and steam pressure lines on the pumping station gages have approached nearer to the outside circles. Higher water pressure has to be maintained on the pumps to keep up the pressure on the distribution mains. A comparison of the gage charts from year to year shows this steady gradual increase of the pumping station pressures, and, at the same time, lessening of the pressure at the office and other points on the distribution system. Finally, the pumps are unable to give a proper pressure in the town, though they have a capacity even greater than the demand. A new force main is installed, and the original conditions are nearly restored, though the pressure lines are not quite as steady as at first, owing to bad condition of the pipe system. A thorough cleaning of the mains and distribution pipes restores the steady pressure line near the outer edge of the circle. So, the gage chart tells us when a new pumping engine is needed, when to install a larger force main and feed mains, when the mains need cleaning.

There is now an instrument for ascertaining from the pressure lines on circular charts, the style now mostly used, the average daily or weekly pressure. These results can be conveniently plotted, so as to show compactly the average pressures throughout a year, or many years. The plotting of two or more years on the same sheet, using different colored inks for the different years, shows compactly the comparative pressures for the equal seasons of each year. Temperature and weather condition notations would add to the interest of these charts. For instance, the month of February in one year shows a steady and good pressure line, while the same month of the next year shows low pressure and a ragged line. The temperature reference shows for one year a moderate temperature, for the other steady cold weather with extremely low temperatures. Without the temperature record the great difference in pressure would be hard to explain.

Consumption records should also be included, to account for the probable steady decrease in average pressures from year to year. Like the February temperature record, the mid-summer temperature and precipitation records would also tell their story.

Pressure recording gages require little care, if daily charts are used, and they give the best records—daily changing of the charts, winding the clock, and filling the pen, are all that is necessary. The supply pipe should be blown out occasionally, to keep it clean, and the clock kept regulated, so that the time records will be accurate. This last item is important, should disputes arise, concerning pressures at time of fires, particularly in towns where an increased fire pressure is required. The rise of the pressure line in the chart should correspond to the instant with the record of the fire alarm. In this connection, some care must be exercised in placing the charts, to see that they start on the correct time line. Some troubles have been caused by carelessness in this, occasionally, for some unapparent reason, the gage chart does not revolve, though the clock is going. This may be due to careless securing of the chart or to putting in two charts at once, the under one revolving all right, but not receiving the impression of the pen or pencil, the outer one slipping on the smooth surface of the lower one and remaining stationary.

The Water Board of Omaha, Neb., has ordered 2,000 meters which will be set during the summer.

Installation and Testing of Double Check Valves Between Public and Polluted Industrial Water Supplies at Auburn, N. Y.

In 1908 an examination of all check valves on mill and factory connections was made by the water department of the city of Auburn, N. Y. These valves were faulty in design and of poor material. In many cases the location of the check valve was unknown. Some had been in service for 20 years without a single examination. Such valves were usually simply buried in the ground. The result of the investigation mentioned was that every industry having a polluted private supply of water was required to install double check valves of a special design, and so set in brick or concrete vaults as to be at all times accessible to inspection and test. The design of these valves, their setting, and the method employed in testing them are here described from information taken from a paper before the American Water Works Association by Mr. J. Walter Ackerman, superintendent of water works at Auburn.

The specifications under which the check valves were made require that the seat ring, disc holding ring, and screws, and in fact all moving parts, and the hinge pin and bushings of same be of bronze, the disc to be faced with medium hard rubber, and liberal clearance to be made all around the clapper, the pin, the arm, etc. These specifications were largely determined by the inspection department of the Associated Factory Mutual Fire Insurance Companies of Boston, and they also had to meet with the approval of the water department of the city of Auburn.

The regulations are that they shall be tested once every six months by the water department and taken apart once each year and examined for defects or obstructions, and during this latter inspection a representative from the Associated Factory Mutual Fire Insurance Company is present to observe the condition of the valve and assist at the test.

It is specified by the water department that the inspection and test shall be at the expense of the factory or mill where the tests are made.

Figure 1 shows a diagrammatic representation of an installation made in the city of Auburn. The stop gate valves *A* and *D* are so placed in line as to shut off absolutely the water from the check valves. On the entering side, between the first valve *A* and the check valve is a tap with cock and air vent, on which is placed the pressure gage *G*. Likewise, two other taps on the two check valves are made with the same type of connection. Tapped into the bottom of each of the check valves is a ¾-in. connection, controlled by cocks *E* and *F*.

The actual test is made as follows: Closing gate *A*, either the cock *E* or the air cock *H* is opened. The pressure at *G* only should fall if the check valve *B* is tight and the gate *A* is tight. It sometimes happens that the opening of air cock *H* releases the pressure so slowly that the back pressure on check valve *B* will not seat the clapper *S* in valve *B*, and the gages both *G* and *J* will fall; while if the ¾-in. cock *E* is opened the valve *B* will close tight. This simply gives the degree of sensitiveness of the check valve, and indicates its condition; for if the air cock does not show it absolutely tight, the drip *E* may. This generally shows that the rubber facing on the clapper is either coated with slime, or some small indentations occurring, allow the water to pass back through this valve.

The conditions of tests are slightly different, depending upon whether a mill has one connection or two. If it has two connections there is always present the city pressure from the other connection coming back to the two valves; whereas, if there is only one connection there is only the static head, or else the fire pumps in the mill are started, and pressure created in that way. If the flow from *E* or *H* is continuous, it is an indication that there is a leak either through both check valves or through the stop gate *A*. If the pressure with *E* and *H* open drops on *G* only, and not on *J* and *M*, it is the gate *A* leaking; but to make absolutely sure, allow all the gages to assume the normal pressure, and then suddenly release *F*. If gages *J* and *G* drop, and *M* does not, and flow continues, then it is an indication that the leak is through *A*, and not through *B* and *G*.

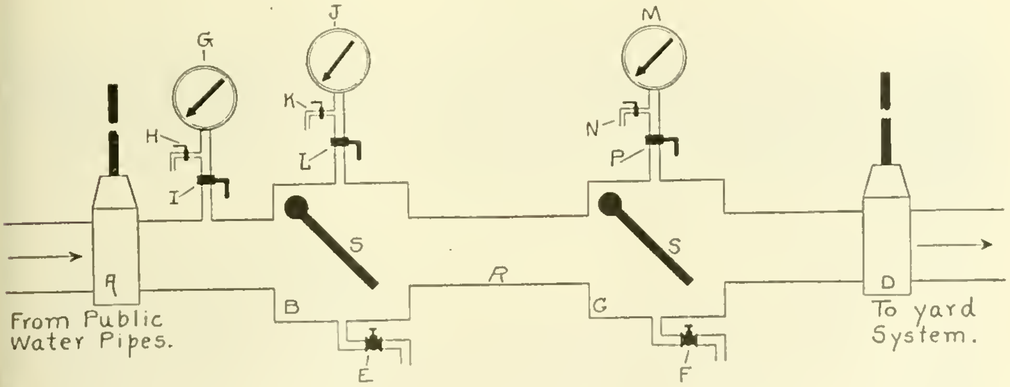


Fig. 1. Sketch Showing Double Check Valve Installation for Separation of Public and Industrial Water Supplies at Auburn, N. Y., and Apparatus for Testing Valves.

After the condition of gate valve *A* and check valve *B* has been thoroughly determined by the above method, the opening of air cock *K* or drip *F* will disclose whether there is a leak on check valve *G*. And, as before noted, if it shows a leak with *K* open, and tight with *F* open, it simply does not seat properly under a low differential pressure. And this is important, because the valve is called upon to act when the fire pump raises the pressure above that of the water in the city mains. And there always will be a time when the pressure on each side is just equalized, and then the pressure on the mill side will begin to build up above that of the city side. And while the test may show from both the drips a tight condition, it will not show a tight condition when only the air cocks are used slowly, allowing the difference in pressure to be gradual.

In order that a number of trials may be made on check valve *B*, pressure should be taken from some source and introduced between the two valves. It is preferable in this case to take the pressure from the main on the mill side between check valve *G* and the mill or factory and allow it to come in through either the drip pipe *F* or some other place, in order to balance the pressure each side of check valve *G*. Then this by-pass connection would furnish water for unlimited trials on check valve *B*, independent of check valve *G*, for any water which leaks through *B* would be replaced by the by-pass connection, which, if it was not there, would soon be exhausted if check valve *G* was tight.

The actual value of the test is obtained after it is completed and the examination made of the interior to show actual conditions, such as tuberculations on the iron body, the smoothness of seat ring, the condition of the rubber facing on clapper, and the manner of the closing of the valve. And while the test is only to show working conditions, the examination shows the way the valves are living up to the requirements imposed upon them.

The rules in the Auburn water department which control the installation of check valves and kindred appurtenances are as follows:

No water pipes in any building or premises supplied with water by the city shall be allowed to receive a supply of water from any other source except in case of pipes used exclusively for fire protection that conform to the requirements given below:

No connection of the city mains shall be allowed with pipes having another source of water used for miscellaneous purposes except to a tank open to the air, nor shall such pipes have any physical connection with the pipes used for fire protection.

Between the pipes used exclusively for fire protection and the city mains there shall be two check valves placed between two positive valves and located in an accessible brick or concrete pit, with such valves, gages and connections as may be prescribed by the water board for each installation and for the purpose of testing. The check valve shall be of a type approved by the water board. The check valves shall be kept in working order by the consumer and shall be at all times subject to inspection and test by the superintendent of water works, the cost of such test being borne by the consumer.

The pure water pipes that have direct connection with the city supply, before the same have passed through the aforesaid double check valves, shall be painted BLUE; the pipes used exclusively for fire protection shall be painted RED; and the pipes carrying the impure water and that used for miscellaneous mill purposes but not for drinking shall be painted YELLOW. These different systems of pipes shall not have any physical connection with each other in the mill or factory.

There shall be placed on the discharge main of the fire pump a check valve and positive gate. The latter shall be kept closed and sealed by the superintendent of water works, and opened only in case of fire or for the purpose of occasionally testing the fire system, but notice of such occasional test shall be given to the superintendent of water works, and during the test the positive gate between the fire system and the city main shall be kept closed. During the weekly trials of the pump prescribed by the fire underwriters the water shall be discharged either through the relief valves of the pump or through hose connections from the pump.

Models Show Proper Lumbering.—Two models which show how the government sells its timber have just been prepared in Washington for display at the forest products exposition, to be held in Chicago April 30 to May 9 and in New York May 20 to 30. These models represent an acre of western yellow pine land in a national forest of the southwest before and after logging.

In the model showing the stand before the lumberman goes into it the trees range from those only a few years old to large, overmature trees, more than ready for the ax. In the second model the mature trees and all others larger than a certain diameter have been cut down and made into logs and cord wood. In this, as in all government sales, the stumps are cut low to avoid unnecessary waste, logs are taken to a small diameter well up into the tree, and such material as is not fit for lumber is converted into cordwood. Together, the models show the care which the government requires of lumbermen in felling old trees so that the young growth is not injured. The brush is piled in heaps for burning after the timber has been removed, in order that the fire menace which foresters say usually follows lumbering may be done away with. The models are on a scale of about one inch to five feet, so that trees approximately 100 ft. tall are about 20 ins. high in the models.

Power Plants

Structural Features and Construction Methods Hydro-Electric Plant at Elkhart, Ind.

Contributed by William G. Fargo, M. Am. Soc. C. E., and Lucius B. Andrus, M. Am. Inst. E. E.

A hydro-electric development comprising some unusual features was completed in June, 1913, on the St. Joseph River, in the city of Elkhart, Indiana. This plant replaces an extensive system of old-time developments made possible by canal systems on both sides of the river, taking water from a stone-filled crib dam with about 12 ft. head. This dam was located about a quarter of a mile above the junction of the Elkhart River with the St. Joseph River. The new plant, Fig. 1, has a head of 18 ft., and the new dam is built immediately below the old one, the concrete work of the new dam being joined to the concrete apron which had been added to the old dam at a later date.

Unusual Features.—(1) The spillway consists entirely of Tainter gates, there being eleven gates 25 ft. wide by 9 ft. 6 ins. high. Three of these gates are steam heated. (2) The headgates are of the Tainter type. (3) The plant is designed for the future building of a steam auxiliary of at least 7,500 kw. initial installation, in immediate connection with the present water power generator house, so that the same crane can serve both generator rooms and a reduction of operating forces can be effected.

At present a portion of the steam turbine room, in line with the generator room, has been built for 60 ft. of its length and provided at the far end with a temporary sheet metal wall. No boiler room is at present built, but certain foundation work has been put in on the downstream side of the turbine room, as well as certain foundations and walls for the future coal storage.

The whole plant has been laid out on the ground, so that a steam railroad spur can be run parallel with the boiler house for future coal supply.

A screen house, with intake pipes and valves, has already been built for furnishing cooling water for the future condensers.

General River Data.—The watershed of the St. Joseph River, at the location of this dam, is about 2,180 square miles. The ordinary summer flow is about 1,300 c. f. s., with a minimum of about 500 c. f. s. The maximum flood of record, in March, 1904, was about 18,500 c. f. s. The present effective pond area, at 18 ft. head, is about 625 acres.

Arrangement of Plant.—The plant is of the open flume type and there are three flumes or penstocks, beside the excited stall. The main flumes are 22 ft. wide and carry 16 ft. of water above the floor. The total length of the penstocks is 81½ ft.

The plant as now built, comprising the penstocks, generator room, switch and transfer rooms, occupies a compact rectangle, 148½x120 ft. The exterior of the buildings is faced with an attractive vitrified brick laid in black mortar. The roof consists of 3-in. reinforced concrete slabs, 2½x5 ft. in size, laid on steel purlins carried by steel trusses. These slabs are covered by green glazed tile.

The main power house is served by a 30-ton hand crane, built by the Northern Engineering Works, Detroit, and the transformer room by a 10-ton hand crane.

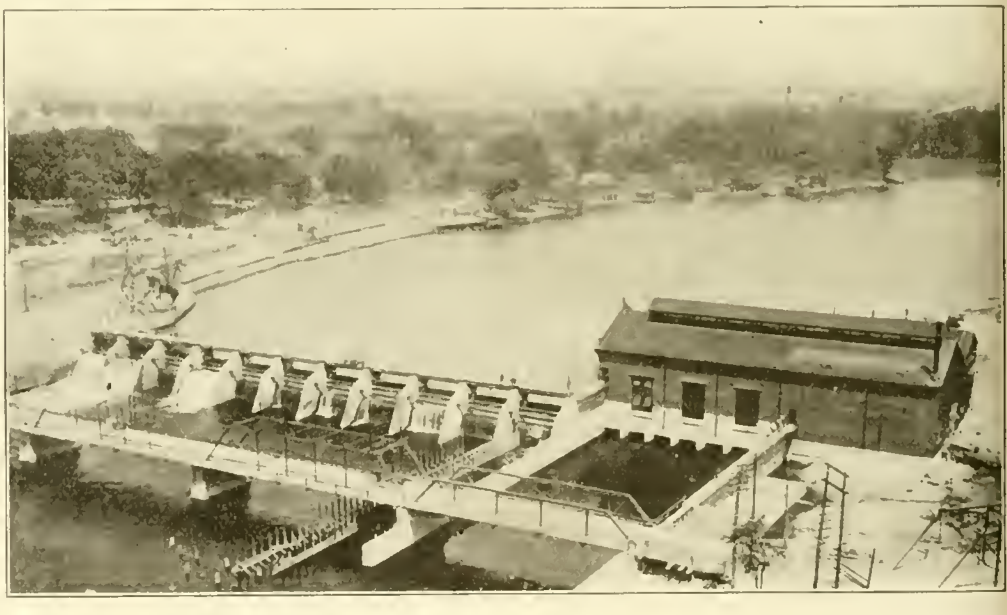


Fig. 1. Hydro-Electric Plant, Elkhart, Ind.

Turbines.—Only two main units and the exciter unit are installed. Each turbine unit consists of two pairs (arranged tandem) of James Leffel & Company's 50-in. horizontal shaft, center discharge turbines, in cast iron draft chests discharging through draft tubes formed in the concrete, each pair of turbines having a separate draft tube of elbow shape. These draft elbows are curved laterally and offset so as to secure the length of draft tube sufficient correctly to reduce the velocity of the water before delivering it into the tailrace. Each of these turbine units, under a net head of 18 ft., is guaranteed to develop 1,400 HP. at full gate opening, at 120 r. p. m.

The exciter turbine unit consists of one pair of 26-in. horizontal shaft, center discharge turbines, developing 170 HP. at 257 r. p. m.

Turbine Test Structures.—As will be seen by an examination of the general plan of the power plant, Fig. 2, there is a tailrace channel approximately 83 ft. wide and some 200 ft. long, leading straightaway downstream from the turbines, and having smooth concrete side walls and for the most part a concrete bottom. This arrangement forms an ideal location for a Francis weir for the purpose of measuring the water discharged through the turbines. The crest of the weir is about 215 ft. downstream from the center of the nearest turbine. The actual length of the weir is 82.52 ft. This weir was formed by placing plank wickets between vertical H-section posts set in a concrete foundation. The wickets were surmounted by an adjustable steel crest with a sharpened edge, according to the Francis construction, and the whole apparatus, except the posts, made removable, so that future tests may be run. The hook gage cisterns, four in number, are all located at the left side of the channel when looking downstream. There are four inlets, two located near the side walls and two at intermediate points across the tailrace. These inlets are connected with the cisterns by a 2-in. galvanized pipe.

On June 3 and 4, 1913, a turbine test was run and the power output from the generators was measured by means of a water rheostat located at headwater level in the unused penstock.

Turbine Lubrication.—The water turbines are provided with a forced grease system for lubricating all of the lignum-vitæ bearings in the water. This system consists of a single large grease compressor in the power house, of capacity sufficient for the two present and one future unit, also the exciter turbines. This compressor is operated by a geared hand wheel and is piped to three cast iron manifolds, to which are attached 1½-in. pipes leading to the bearings in the water. On these pipes, at convenient places in the power house, are quick-opening valves. All bends are long radius. The pipes are laid over the floor of the penstock and are securely anchored thereto, and from thence run on the sides the cast iron draft chests or pedestals to their respective bearings.

The water wheel bearings are lubricated by opening the valves successively and turning the hand wheel on the grease compressor. From one-half to a full turn is all that is necessary for supplying the amount of grease needed twice a day. After turning the hand wheel the valve is closed and the next valve opened, and this procedure followed until all bearings are lubricated. The lignum-vitæ bearing housings in the water are so made that if the grease lubrication should fail for any reason, such as the breaking of one of the feed pipes, there will be sufficient water reach the bearing to prevent its burning out; but while the grease compressor is on the water will be kept out of the bearing chamber.

The shifting rings for operating the wicket gates of the turbines are also provided with grease lubrication at intervals around the upper portion of the rings.

Generators.—Each of the main turbine units is directly connected to a 1,000-kva., three-phase, four-wire, 4,000-volt, 60-cycle generator unit. The exciter generator is 100-kw., 125-volt, direct-connected. All were built by the Westinghouse Electric & Manufacturing Co.

Speed Regulation.—The generator specifications called for a fly wheel effect equivalent to 1,000,000 foot square pounds at 1 ft. radius. This had the effect of requiring a heavy spider in the rotating element of the generator and heavier shaft bearings, etc., but the better regulation obtained by the careful attention to fly wheel effect, not only in this plant but the other plants in the system, has resulted in very excellent regulation.

Turbine Governors and Connections.—Each turbine unit is equipped with a Lombard horizontal oil pressure governor, 30,000 ft. lbs. capacity, each having its separate oil pump and pressure tank. Each main unit governor is fitted with an electric speed control, operated from the switchboard. These governors are of the direct-connected type and are placed on high cast iron stands so as to straddle the main gate operating shaft, which is carried on top of the turbine cases.

The exciter turbine is regulated by a Lombard horizontal oil pressure governor of 2,500 ft. lbs. capacity, and is also furnished with a separate oil pump and pressure tank.

Spillway.—The present concrete spillway consists of eleven steel Tainter gates and replaced an old timber crib dam of triangular section, which had a total length of 291.2 ft. The new spillway was designed to have equal or greater discharging capacity without increasing the depth of water over the crest, but was shortened to a net length of 275 ft. The Tainter gate sills are 0.35 ft. lower than the crest of the old dam.

The one disadvantage of movable dams in a northern climate is the fact that they are likely to freeze tight in winter. To prevent this trouble, the central three Tainter gates were provided with steam chambers along their sides and bottom, and curved steel steam chambers, corresponding to the shape of the gates, were embedded in the piers between the gates. A low pressure heating system was installed in the power house and well-insulated pipes carried over the gates on the deck or bridge, with connections to the chambers in the piers and swinging connections by means of pipe joints and short lengths of rubber hose to the chambers in the gates themselves. Similar heating devices have been installed in connection with other Tainter gate spillways designed by the Fargo Engineering Co., and all have proven successful.

The foundation soil under the spillway is a compact bluish clay in which were horizontal laminations caused by 1/16-in. or thinner fine sand seams at 1½ or 2-in. intervals. These fine sand seams were exactly horizontal and so thin that under any heads existing in the cofferdams but little seepage was apparent. These thin sand seams would not be noticed except for a weathered effect on vertical or sloping banks, which had the appearance from a little distance of the leaves of a book.

The new dam was built at the downstream side of the old crib dam 12 ft. in height, which thus became an element of safety in sealing the bottom upstream from the new structure. The new dam was joined and sealed to the concrete apron which had been put in below the old dam by means of a grouted rubble fill, which in general reached a level 7 ft. below the sills of the Tainter gates, or about 15 ft. below headwater. Every part of the base of the new structure was placed on undisturbed clay in which were no seams except the thin horizontal sand seams above mentioned. Deep toothing trenches were cut into the clay at the upstream and downstream margins of the main spillway base. Downstream from the spillway base proper an apron, consisting of book slabs, in general 16½ ft. by 17 ft. 4 ins. in size, extended for a distance of 115 ft. The lower two rows of slabs, however, were made of riprap stone and grouted.

Concreting Methods.—The total amount of concrete used in the Elkhart dam and plant is 20,022 cu. yds. In this plant and protection walls built upriver from the plant there were used a total of 29,130 bbl. of Wolverine cement. The total amount of reinforcing steel used in the dam and plant was 616,500 lbs.

For the most part the concrete aggregate consisted of gravel which was obtained within a few hundred feet of the work, so that no hauling was required. This gravel contained approximately the proper proportion of sand for making a dense concrete, and for heavier mass portions of the dam and other parts of the work was used without screening or washing. All of the thinner walls of the dam, which were built during the winter of 1911-12, were made from a mixture of broken stone and gravel aggregate in varying proportions so as to secure the greatest density, and with an amount of cement not exceeding 1¼ bbl. to the cubic yard of aggregate.

As soon as the weather permitted, in the spring of 1912, a washing plant was installed on the right bank of the river and within 200 ft. of the end of the dam. The gravel bed lay immediately adjacent to this washing plant, and the gravel was excavated and delivered within reach of a derrick by means of a scraper bucket on an endless cable. The sheave at the far end being attached to a trunnion [illegible] thus giving a fan-shaped area to the pit. The gravel was hoisted by means of a derrick and clam shell bucket to the receiving hopper of the washing plant, from which it was washed by a stream of water from an ordinary [illegible] centrifugal pump. A trough 1 ft. 6 ins. wide and 12 ins. high lined with sheet iron, led on a 7 per cent grade from the receiving hopper to the sorting hoppers, the first of these hoppers taking the heavier gravel and the second the sand, the silt passing along with the water.

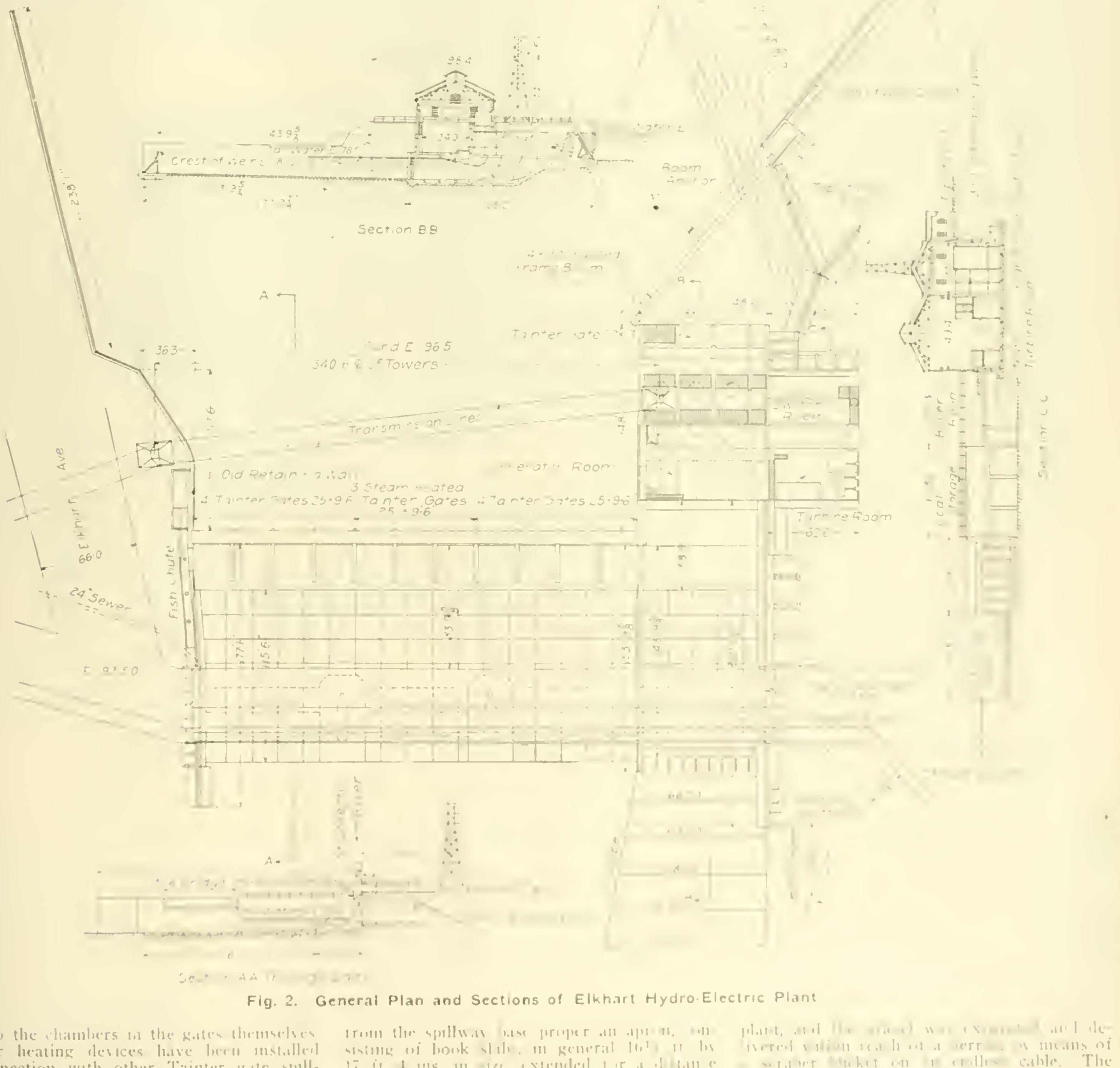


Fig. 2. General Plan and Sections of Elkhart Hydro-Electric Plant

As the amount of sand in the natural gravel was quite uniform, and by frequent tests was found to be the proper amount to produce dense concrete, it was feasible to remove the partition between the sand and gravel pits and discharge both of the sorting hoppers into a common bin. The cost of excavating and washing of the gravel, including plant depreciation, but without overhead, such as office charges, was about 18 cts. per cubic yard.

The mixers were placed close to the washer and the mixed concrete was delivered to the work by means of ½-cu. yd. side dump cars running on 24-in. tracks, the maximem length of run being about 550 ft. to the extreme side of the power house. The mixers were located on the right side of the river at the end of the dam, consequently the average haul for concrete in the dam itself was about 150 ft.

In the power house section the following is a fair example of the labor cost of digging and washing gravel, mixing and placing concrete.

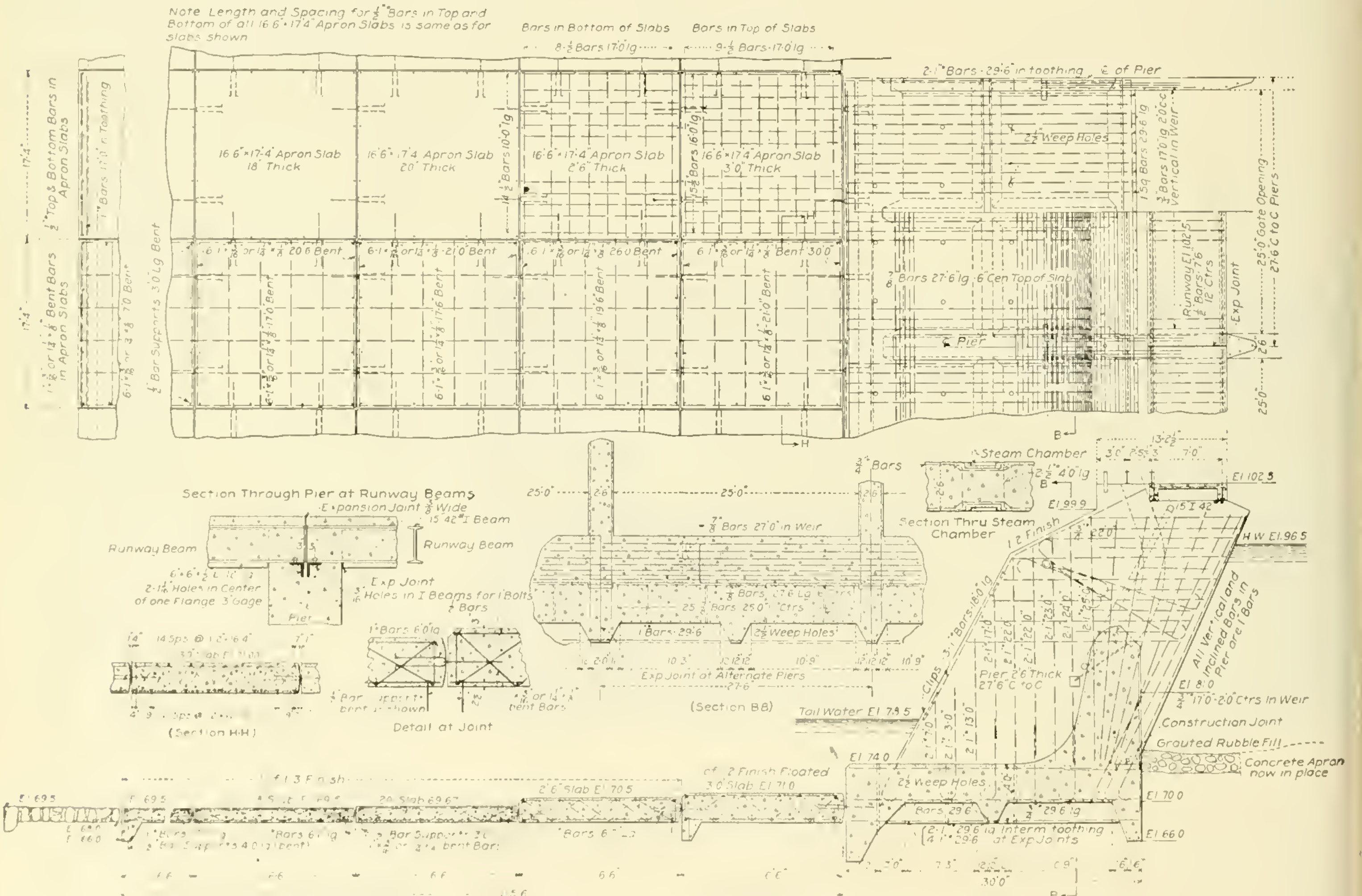
	Yards.	Total Cost.	Cost per Yard.
Washing Sand and Gravel	1,690	$299.64	17.7
Feeding Mixer	1,530	260.17	17.0
Operating Mixer	1,530	79.94	5.2
Running Concrete Cars, 550 ft.	1,530	222.77	14.6
Labor in Forms		244.58	16.0
General, Interpreter, Foreman, etc.		151.08	9.9
Total per yard			80.4

NOTE: No overhead included. Cost of putting on tops included. It should be noted that common labor was paid $2.25 per day, and that the work was run short handed throughout and at a much diminished efficiency.

A peculiar appearance of vertical sand streaks on some of the concrete walls was observed on stripping the forms. These flow markings only appeared on about one-half of the work. They consist of rather fine round sand, not coated with cement, and consequently after the removal of the forms this sand could be rubbed off with a stiff brush. The depth of this loose sand seldom exceeded 1/16 or ⅛ in. The direction of flow was upward and, after extensive efforts to determine the cause, the following seemed the most likely explanation:

The forms where these sand streaks appear were uniformly smooth lumber, well oiled, or were lined with steel. The concrete was put in quite wet and was perhaps puddled with tools a little more than was absolutely necessary. It may or may not be true that some of this sand would repel the cement, but in puddling the effect was for this sand to travel up against the forms, as this was apparently the path of least resistance. On the first appearance of these sand streaks, chemical and physical tests were made in different laboratories to determine whether there was any likelihood of disintegration of the concrete due to the primary causes of the streaks. Nothing of this sort was found. The concrete back of the streaks was perfectly hard and sound and has remained so. Somewhat drier concrete, or concrete which was not worked with tools at all, showed the same effect, but in a lesser degree. The surfaces back of these streaks are left granulated and without the smooth, hard skin which it otherwise would have had. In the concrete draft tubes it was thought necessary to do some surface coating and plastering to avoid this disintegration, owing to the granulated surface left by some of these streaks. A few of the retaining walls were floated with a thin grout to better the appearance. As before mentioned, many of the walls show but little evidence of such streaks.

The Elkhart dam and plant was built on force account by the owner, Mr. F. A. Bryan, Vice-President and General Manager of the Indiana & Michigan Electric Co., giving personal attention to the work, which was in charge of Mr. B. J. Campbell during the fall and winter of 1911; and later, following Mr. Campbell's illness and death, was completed by Mr. D. L. Davis, who was responsible for the general layout of the gravel washing and concreting equipment. The electrical design and construction was in charge of Lucius B. Andrus, Mem. Am. Inst. E. E., General Superintendent and Electrical Engineer for the Indiana & Michigan Electric Co. The Fargo Engineering Co. of Jackson, Mich., designed the dam and the structural and hydraulic features of the plant, and their resident engineers were Messrs. D. N. Showalter and H. M. Morse.



Note: Length and Spacing for ½" Bars in Top and Bottom of all 16'6" × 17'4" Apron Slabs is same as for slabs shown
Bars in Bottom of Slabs
Bars in Top of Slabs
8-½" Bars 17'0" lg
9-½" Bars 17'0" lg
16'6" × 17'4" Apron Slab 18" Thick
16'6" × 17'4" Apron Slab 20" Thick
16'6" × 17'4" Apron Slab 2'6" Thick
16'6" × 17'4" Apron Slab 3'0" Thick
2-1" Bars 29'6" in toothing
℄ of Pier
2½ Weep Holes
Runway El. 102.5
Exp. Joint
25'0" Gate Opening
27'6" C to C Piers
Bars 27'6" lg 6" Cen Top of Slab
Pier
Section Through Pier at Runway Beams
Expansion Joint ⅜" Wide
15 42# I Beam
Runway Beam
Exp. Joint
Steam Chamber
Section Thru Steam Chamber
El. 102.5
El. 99.9
H.W. El. 96.5
⅞" Bars 27'0" in Weir
2½ Weep Holes
Exp. Joint at Alternate Piers
(Section BB)
(Section H-H)
Detail at Joint
Tail Water El. 78.5
Pier 2'6" Thick 27'6" C to C
All Vertical and Inclined Bars in Pier are 1" Bars
El. 81.0
½" 17'0" 2'0" Ctrs in Weir
Construction Joint
Grouted Rubble Fill
Concrete Apron now in place
El. 74.0
El. 70.0
El. 66.0
2½ Weep Holes
2-1" 29'6" lg Interm toothing
4-1" 29'6" at Exp Joints
El. 69.5
2'6" Slab El. 70.5
3'0" Slab El. 71.0
1:2 Finish Floated
1:3 Finish
30'0"
B
H

Fig. 3 Details of Concrete Work for Tainter Gate Dam, Elkhart Hydro-Electric Plant.

Method and Cost of Constructing the Huacal Dam in Sonora, Mexico.

The Huacal Dam is built across the canyon of Huacal Creek, about three miles northeast of Nacozari, Sonora, Mexico, at an elevation of 550 ft. above the town and 4,100 ft. above sea level. Nacozari formed the base of construction supplies. Bulky pieces were transported by wagon over a rough mountain road some six miles in length, with steep gradients, the maximum load being one ton to four good animals. Everything else, including all cement and sand, was packed on mules and burros, with loads of 300 lbs. or more to the mule and 175 lbs. to the burro. The average cost of packing was about $1.02 per 2,000 lbs. The cement was packed from the railroad depot, a distance of about six miles, and the sand from the Nacozari River about four miles. The cost per ton mile one way, averaged about 21¾ cents. All prices and costs are given in United States gold values. An account of the design and construction of this dam is given in a paper by Mr. H. Hawgood, Proceedings American Society of Civil Engineers, Vol. XL, p. 925, from which we take the following information on construction methods and cost:

DESIGN OF DAM.

The narrowness of the canyon and the character of its rock sides and floor suggested an arch dam as the economic type for the location. Comparative studies with a rock-fill dam as a practical alternative confirmed the selection of the arch type. The requisite cross-section of the dam was determined by the simple formula for cylinders subjected to external pressure:

$$\text{Stress, in tons (2,000 lbs.) per sq. ft.} = \frac{\text{Radius (ft.)} \times \text{Pressure (lbs. per sq. ft.)}}{\text{Thickness (ft.)}}$$

The values finally adopted in the design of the dam were: Stress, 16.3 tons per square foot; radius, 76 ft.; and the equation for thickness became $T = \frac{76}{16.3} \times \frac{D}{32}$, whence the thickness T at the depth $D = 0.146\ D$.

With these values the theoretical triangular dam section has a vertical front face, and a back face on a batter of 1¾ ins. to the foot. The lower portion of the dam is built of this form, the upper portion being widened to meet the practical necessities which govern a dam top.

DAM STRUCTURE.

The extreme height of the Huacal Dam from base to crest is 100.25 ft. The foundation trench was excavated until sound rock was reached at depths from 5 to 15 ft. That there might be no powder cracks, hand tools only were used in the bottom of the trench. Similar precaution was exercised in the abutment trenches at each end of the dam, which were carried horizontally into the rock formation to a maximum distance of 20 ft. to insure contact with sound rock.

The front face of the dam is vertical, the back face has a variable batter changing from 1¾ ins. to the foot at the bottom to ½ in. to the foot at the top, as shown on the cross-section, Fig. 1. As actually built, the back face does not conform with this design precisely. By a mishap, the dam about its middle height is somewhat thicker than designed. The profile was returned to the true lines with an easy batter, as shown in the drawing.

The top of the arched portion of the dam is made hollow, both to save concrete and to provide free intercommunication between the space beneath the sheet of falling water when the dam is overflowed and the open air, and thus avoid the tremors incidental to the making and breaking of contact between water and dam with the forming and breaking of a vacuum behind the nappe when the free passage of air is wholly or partly suppressed.

The dam is not of the arch form for its entire length. For 42 ft. at its southeasterly end it is of gravity section, and in direction tangential to the curve.

The continuance of the dam on a curve throughout its entire length would have brought the direction of its easterly end too closely parallel to the axis of the canyon to have been economical or safe.

In the canyon below the main structure there is a supplementary dam designed to create at times of overflow a water cushion with a maximum depth of 20 ft. over the base of the main dam.

CONCRETE

The writer's previous experience having shown a 1 to 2 mortar to be safe for water-tight concrete, and that a 1 to 2½ mixture bordered on the pervious, a proportion of 1 of cement to 2¼ of sand was adopted, giving an excess of about 25 per cent of cement over the voids in the sand. The coarse aggregate was crusher run, graded from fragments of ¾ to 1 in. down to a coarse, sharp sand. The average of numerous tests gave the void spaces of the crusher-run aggregate, after compaction by shaking and jarring, as 38 per cent.

A series of tests was conducted to determine the proportion of sand to crusher run which would give a mixture of greatest density. These tests were made by progressive weighing of a box of known volume filled with the mix to be tested, the box being shaken and jarred during the process of filling. The greatest average weight was attained with 1 of sand to 2.03 of crusher run, the void space being 19 per cent.

The proportions actually fed to the mixers averaged 1 cement; 2 sand; 3.85 crusher run. The sand of the crusher run added to the straight sand made the desired 1 to 2¼ mortar. The proportion of sand in the crusher run varied, of course, from time to time, with the wear and changing and consequent slacking and tightening of the crusher jaws. The foreman watched the composition of the crusher run closely, and from his observations adjusted from time to time the sand component so as to maintain practically constant the desired ratios.

The resultant mix gave a mortar approximately from 35 to 40 per cent in excess of the voids in the coarse aggregate. Density of concrete was obtained with a sufficient surplus of mortar to coat thoroughly the surfaces of the 20-lb. to 300-lb. "plum" stones which were introduced as liberally as practicable. The "plums," first well wetted, were dropped into the mush concrete and worked around to insure liberation of any imprisoned air. The "plums" constitute probably 15 per cent of the entire mass of concrete, a larger percentage would have been added if the hoisting facilities had permitted. The consumption of cement, as a whole, averaged about 1.3 bbls. per cubic yard of completed concrete. Samples of finished concrete broken out of the actual structure weighed, dry, 152 lbs. per cubic foot.

The concrete has proved to be water-tight to a marked degree, such slight dampness as appears in spots on the back face of the dam being more probably due to seepage at overnight joints between one day's work and another than to percolation through the body of the concrete.

PLANT.

The concrete plant was erected on the east side of the canyon at a higher elevation than the top of the dam. Cement was unloaded directly from the pack animals into store at the highest point of the plant, and the transportation was paid for by count of the sacks delivered. Sand, with transportation paid for by delivered weight, was unloaded and weighed into a bin near the bottom of the canyon and hoisted by bucket on inclined cable and delivered into gravity bins. The rock crusher was operated at an elevation corresponding with the spillway floor and fed by rock from the spillway excavation. The product of the crusher was raised by belt and bucket elevator to the bins.

Cement, sand and crushed rock gravitated from their respective bins to the batch-measuring hoppers, and thence to two mixers, one with a capacity of 12.7 cu. ft. to the batch, and the other of 21.7 cu. ft. The average composition of the mix was 1 cement; 2 sand; 3.85 crusher run.

The mixers delivered into a 1½-cu. yd. bucket slung between two standing cables, one crossing the canyon immediately below the dam and the other immediately above. Traveling carriages, one on each cable, moved in and out together. Hoisting tackle, independently operated, reached from each traveling carriage to the bucket bail. This arrangement permitted the depositing at will of the bucket at any point on the dam. Traveling and hoisting ropes were operated at speeds of about 600 ft. per minute light and 400 ft. per minute loaded. The maximum quantity of concrete placed in any one day was 114 cu. yds. and the average per day for the days on which the plant was running was 37.5 cu. yds. The minimum working time of mixing and placing 1 cu. yd. of concrete was 3.2 minutes, the maximum was 19.25 minutes per cubic yard (refilling outlet tunnels), with an average working time of 8.7 minutes per cubic yard for the whole work.

The machinery, picked up around the company's mines and not without some incursions into the scrap pile, was of a heterogeneous character, incompatible with compactness or efficiency of operation. A saving in plant investment was effected at the expense of the constructional cost of the dam.

There were five steam boilers, ranging from 10 to 35 HP., supplied with feed water by a direct-acting duplex steam pump, drawing from water impounded above the dam. Six engines were used, a 25-HP. for hoisting and a 30-HP. for traversing the carriages on the

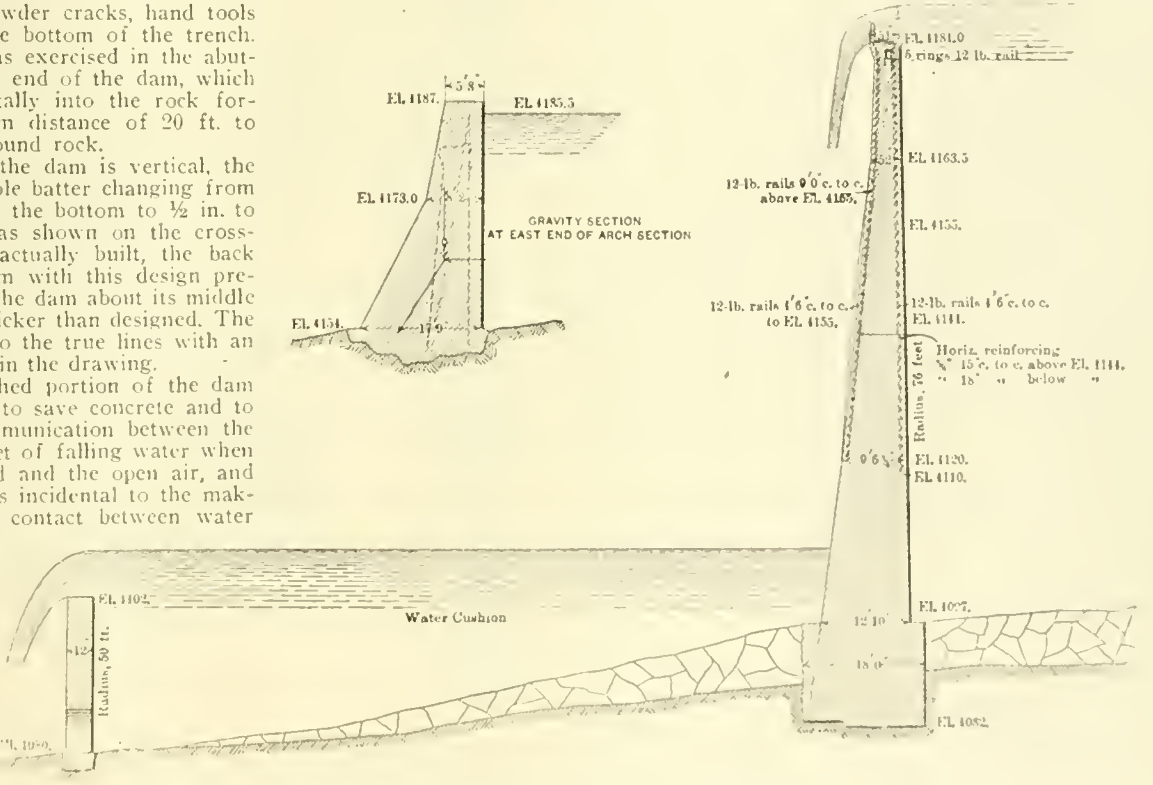


Fig. 1. Sections of Main and Cushion Dams, Huacal, Mexico.

cableways, a 26-HP. on the rock crusher, a 30-HP. on the sand hoist, and 2-HP. engines driving the concrete mixers. The fuel supply was chiefly wood from the live oaks cleared from the reservoir, liberally helped out with coal to compensate for the wood being green.

It would have been feasible to operate by electric current transmitted from the copper company's generating plant at Nacozari. This method was contemplated originally, but at the time was not deemed best. The experience of the work showed that the elimination of boiler and enginemen troubles and the elimination of transportation of coal, and other economies which would have followed with electricity as the motive power, would have resulted in a lower cost of construction.

The work was carried on during the Madero Revolution, when the country was in a state of turmoil; nearby collisions between the opposing forces were frequent, and the labor supply was disturbed and uncertain.

FORMS.

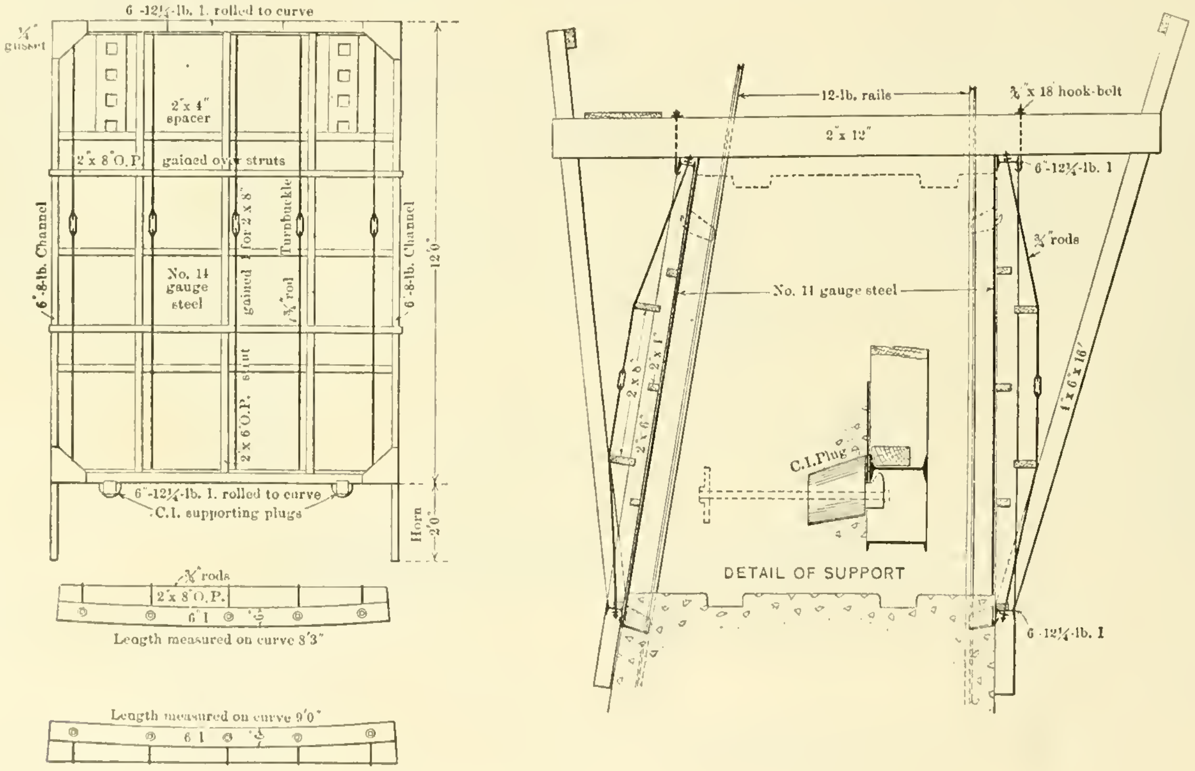


Fig. 2. Steel Forms for Constructing Huacal Dam, Mexico.

The passing of any bolts, wire or other form fastening through from face to face of the dam was prohibited. The lower portion of the dam was constructed with wood forms of the customary stud and sheeting type, to a height above which the external struts bracing the studding would have become inconveniently long. The upper 60 ft. of the dam was built with steel forms (Fig. 2). These were in units 9 ft. wide for the water face and 8 ft. 3 ins. for the back face, and 12 ft. high. They were generally used in threes, making a block space for pouring concrete 27 ft. by 12 ft. and of the width of the dam.

No attempt was made to vary the radius of the back face forms to conform to increasing radius due to decreasing thickness of dam with height. The forms were shaped to a compromise radius. The slight angle made by the adjacent forms, where not strictly conforming to the true radius, was imperceptible on the finished work. The location of the dam and its commercial use did not warrant the increased cost of forms of variable radius which would have been necessary for precision of shape. The front face of the dam being vertical, its radius is constant. The front forms, being first set accurately to position, constituted a base from which were set the back forms.

The steel forms were supported by their lower edges engaging in hook-shaped projections on taper cast iron plugs built in advance into the concrete as the work progressed. The plugs retained their position by virtue of being inclined downward into the concrete. After use the plugs were withdrawn, to be used again, and the pockets were filled with concrete. Two plugs were used to form a unit, one under each corner. The plan proved satisfactory, except in some instances where the concrete bucket with its load struck heavily the top of the form and jarred the plugs loose. However, no instance occurred where the plugs actually let go. A bolt might be added to the plug, as shown by the broken lines on Fig. 2, which could be removed with the plug, the washer alone being lost in the concrete.

The walkway on top of the forms fulfilled admirably its purpose of a safe and convenient working platform for the concrete gang.

The forms were handled into place by the overhead cableways. The cost and time of removing the steel forms from set concrete and placing them in new position was low and in marked contrast to the cost and time of stripping the ordinary wooden forms. After the concrete work was completed the steel farms were dismantled and part of their structural shapes and turn-buckles were used for the spillway gates. The remainder of the metal work was found useful, and absorbed for odd jobs by the mine mechanical department. There was but little material sent to the scrap pile, whereas with wooden forms there would have been practically no salvage.

REINFORCEMENTS.

Both faces of the dam are reinforced vertically and horizontally against temperature stresses. The vertical members are old 12-lb. mining rails, 4 ft. 6 ins. from center to center on the front face, and on the back face 4 ft. 6 ins. from center to center for the lower 35 ft. of their length, and 9 ft. from center to center for the upper 26 ft.

The horizontal reinforcement consists of five rings of 12-lb. rail immediately below the crest of the dam, and, from there down, ⅝-in. round and square rods, with spacing varying from 15 to 18 ins. from center to center. The lower 30 ft. of the dam is not reinforced.

COST.

The volume of concrete in the dam works was 4,988 cu. yds. The cost of the work was as follows per mile cubic yard of concrete:

Labor on roads and trails	$0.306
Plant—	
Buildings, material	0.597
Buildings, labor	0.161
Total	$0.758
Cableways, material	0.315
Cableways, labor	0.147
Total	$0.462
Machinery and supplies	0.873
Installing machinery (labor)	0.796
Transporting machinery	0.238
Water tanks	0.032
Total	$1.939
Total Plant	3.159
Miscellaneous, materials	0.479
Miscellaneous, labor	1.574
Total	$2.053
Gates, central jettings, tower	0.160
Installing piping, tower, labor	0.176
Total	$0.336
Excavation	1.062
Concrete—	
Cement at Nacozari	5.368
Transportation of cement	0.569
Total	$5.937
Sand, screening and handling	0.601
Sand, transportation	1.686
Total	$2.287
Forms—	
Lumber	0.942
Steel	0.520
Labor	1.230
Transportation	0.238
Total	$2.930
Mixing and placing concrete and crushing rock, labor	1.308
Fuel for above	0.727
Blacksmithing, labor	0.250
Blacksmithing, materials	0.082
Total	$0.332
Reinforcing material	$0.291
Superintendence	0.558
Engineering field	0.563
Engineering plans and supervision	0.734
Total	$1.297
Grand total	22.58

In recomputing the above items to get units for all items instead of for groups of items, as did the author, a slight discrepancy exists between the author's total of $22,912 and the total given. The author divided total costs by 4,988 cu. yds., while to reduce numerical work we took even 5,000 as the divisor.

Probably no city of its size in the country has more paving work under contract at the present time than Terre Haute, Ind. On April 6 this city let contracts amounting to $570,000, and on May 12, to $366,000.

General

Volume and Cost of Construction Work on the Panama Canal to Jan. 1, 1914.

The total construction expenditure on the Panama Canal from the beginning of work to Jan. 1, 1914, has been $197,965,955. This total does not include charges for sanitation, hospitals and civil administration nor expenditures for purchase of the Canal Zone or the old French canal company's property. The items which make up this total are as follows:

	Items:	Quantities, cu. yds.	Amount.	Unit cost.
	From and including Gatun to the Sea—			
1	Dry excavation, prism	2,181,998	$ 1,471,285.43	$ 0.6743
2	Hydraulic excavation, prism	29,605	11,671.32	.3942
3	Dredging excavation, prism	38,455,650	8,941,736.62	.2325
	Gatun spillway—			
4	Dry excavation	1,544,202	$ 1,099,661.65	$ 0.7121
5	Preparing foundation	44,715	98,757.88	2.2086
6	Masonry	230,777	1,846,364.04	8.0006
7	Ironwork	....	168,761.01	.....
8	Gates	....	95,526.30	.....
9	Operating machinery	....	104,438.57	.....
10	Backfilling	36,923	18,255.84	.4944
	Total Gatun spillway	....	$ 3,431,765.29	.....
	Gatun dam—			
11	Dredging excavation	38,425	$ 20,041.19	$ 0.5216
12	Dry filling	12,190,415	5,309,231.83	.4355
13	Hydraulic filling	10,728,965	3,202,567.36	.2985
14	Paving excavation	15,075	35,555.78	2.3581
15	Paving riprap	78,590	67,073.08	.8535
16	Paving broken stone	15,740	40,652.92	2.5828
17	Surfacing (square yards)	390,065	13,904.58	.0356
18	East Saddle dam, dry filling	4,117	1,803.89	.4382
	Total Gatun dam	....	$ 8,690,830.63	.....
	Gatun locks—			
19	Dry excavation	4,660,055	$ 3,155,572.44	$ 0.6771
20	Dredging excavation	1,756,977	561,923.85	.3198
21	Preparing foundations—excavation	228,376	453,999.02	1.9879
22	Preparing foundations—dredging	19,814	39,720.50	2.0047
23	Preparing foundations—filling	41,661	16,514.47	.3964
24	Preparing foundations—piles, concrete (lin. ft.)	83,670	141,743.12	1.6941
25	Preparing foundations—piles, wooden (lin. ft.)	251,199	164,235.99	.6538
26	Masonry, mass	2,041,175	15,235,577.36	7.4641
27	Masonry, machine installation	25,531	327,436.95	12.8251
28	Ironwork	....	1,360,954.92	.....
29	Gates and fender chains	....	2,515,134.28	.....
30	Emergency dams	....	855,772.14	.....
31	Operating machinery	....	2,847,177.77	.....
32	Control house, masonry	658	24,707.35	37.5492
33	Control house, ironwork and miscellaneous	....	26,769.97	.....
34	Control house, machinery installation	....	21,586.83	.....
35	Buffer timbers	....	16,809.37	.....
36	Crib fenders	....	38,849.65	.....
37	Backfilling	2,119,168	1,163,003.13	.5488
38	Filling center wall	113,163	90,759.66	.8020
39	Filling around south approach wall—dry	7,072	3,777.61	.6342
40	Filling around south approach wall—hyd	594,495	94,077.49	.1582
41	Cleaning up	....	33,186.12	.....
	Total Gatun locks	....	$29,189,289.99	.....
	Gatun hydroelectric power plant—			
42	Dry excavation	87,067	$ 35,310.45	$ 0.4056
43	Preparing foundations	11,709	23,860.05	2.0378
44	Building construction	....	234,991.21	.....
45	Operating machinery	....	274,639.44	.....
	Total hydroelectric power plant	....	$ 568,800.15	.....
	Gatun-Mindi levee—			
46	Dry filling	263,669	$ 103,008.84	$ 0.3907
47	Hydraulic filling	20,398	3,483.32	.1708
	Total Gatun-Mindi levee	....	$ 106,492.16	.....
48	Caño saddle dam	....	$ 3,559.71	.....
49	Colon breakwater—west	1,206,164	3,376,296.34	$ 2.7992
50	Clearing drift from Gatun Lake	....	7,326.35	.....
	Total construction cost, Gatun to sea	....	$55,799,054.99	.....
51	Plant and equipment to be absorbed after Dec. 31, 1911	....	42,870.34	.....
	Total expenditures, Gatun to the sea	....	$53,756,184.65	.....
	Gatun to Pedro Miguel—			
52	Dry excavation—prism	109,709,781	$85,765,657.64	$ 0.7818
53	Dredging excavation—prism	1,069,788	513,503.37	.4800
54	Hydraulic excavation—prism	732,074	173,764.40	.2374
55	Clearing canal line without excavation (acres)	2,530	157,569.83	62.2[illegible]6
56	Masonry	1,271	8,353.66	6.5721
57	Facing—cement gun, square yards	4,250	4,079.11	.9598
	Total construction cost, Gatun to Pedro Miguel	....	$86,622,928.01	...
58	Plant and equipment to be absorbed in construction costs after Dec. 31, 1913	....	360,316.92	
	Total expenditure Gatun to Pedro Miguel	....	$86,983,244.93	
	From and including Pedro Miguel to the sea—			
59	Dry excavation—prism	4,819,969	$ 3,507,516.41	$ 0.727[illegible]
60	Hydraulic excavation—prism	1,549,904	1,120,907.35	.7232
61	Dredging excavation—prism	39,815,319	10,161,446.73	.255[illegible]
	Pedro Miguel dams—			
62	Dry excavation	10,475	$ 20,125.17	$ 1.9213
63	Dry filling	299,518	336,778.70	1.1244
64	Masonry	...	8,411.34	.
	Total Pedro Miguel dams	....	$ 365,314.71	.
	Pedro Miguel locks—			
65	Dry excavation	1,133,280	$ 1,033,571.[illegible]	$ 0.9[illegible]1[illegible]0
66	Preparing foundations	175,987	479,416.41	2.72[illegible]
67	Masonry mass	9[illegible]7,136	5,342,711.[illegible]	5.88[illegible]
68	Masonry, machinery installation	16,194	198,532.86	12.[illegible]97
69	Iron work	....	666,630.44	..
70	Gates	..	1,566,51[illegible].69	.
71	Emergency dams	....	678,796.28	.
72	Operating machinery	..	1,525,118.9[illegible]	
73	Control house, masonry	797	5[illegible],149.[illegible]9	63.[illegible]87
74	Control house, iron work and miscellaneous	....	22,493.75	.....
75	Control house, machinery installation	...	21,511.80	.....

An Electric Power Railway-Wrecking Crane for Tunnel Line.

(Contributed.)

A noteworthy piece of equipment for use in case of accidents at the Grand Central Terminal has been received recently in New York. This appliance is a double end electric wrecking crane, with independent 100-ton capacity cranes at each end, the whole being especially designed for the underground clearances and conditions existing in Grand Central terminal. While primarily designed to meet Grand Central terminal conditions, it is also adapted for use on the main line of the electric division, where operating conditions and clearances are such as usually obtain on steam roads. This machine may be dispatched at high speed under its own power to the scene of an accident anywhere on the electric division. In fact, it combines the functions of a crane with many of those of a high power electric locomotive. It is completely equipped for high speed, independent operation, with air brakes, whistle, and all necessary fittings.

To design a crane of the large capacity required under the unusual limitations imposed by the conditions of tunnel operation was not a simple matter. In the restricted space of the express level deck of the terminal the capacities must be attained under conditions where head room for raising the boom and room at the side for slewing were both restricted, and, in addition, where excessive concentration of wheel loads must be avoided. In spite of these complications a crane has been built and furnished, fulfilling all the requirements desired. Just as operating conditions in Grand Central terminal are different from those found elsewhere, so is this crane, which was designed to meet those conditions, unusual in its features. The procedure in case of a wreck underground would be somewhat as follows: If possible the wreckage would be lifted clear of the track and the crane would then back out with it. However, if the crane could not raise the damaged equipment off the track—either on account of lack of head room or because of the size of the piece—one end would be lifted with the main hoist and with the auxiliary hoist a special truck would be placed in under. The crane would then drag away the load.

In construction the crane is unusually massive and of great strength. The car body is 67 ft. long, with a wheel base of 51 ft., and is carried on two compound trucks, made up of two four-wheel trucks each. A cradle on which the car body rests allows the compound trucks, as well as each single truck, to have perfect freedom to swing when taking curves. Complete air brakes are provided; also a hand brake wheel at each end There is a comprehensive system of air-operated telescopic outriggers or jack beams, used to add stability during heavy lifting and to distribute the load over a greater area. These are controlled by valves with suitable levers, and are instantaneous in action, thus combining ease and speed of operation.

At each end of the car is a complete independent crane, with a structural mast and boom, accurately turned roller path and rollers, and the slewing mechanism. All of the motions of operation may be performed independently, and with loads up to the capacities of the motors may be performed simultaneously. A special feature is the air operation of all clutches to insure quick and sure engagement and release Such a system with suitable levers is at both ends of the car, each operating its respective end, except for the propelling which may be controlled from either end.

The electrical equipment is noteworthy, not only on account of the power it develops, but because of its flexibility Six principal motors with a total of 1,100 H.P. are used for operating the crane, as follows: Four of 200 H.P.

	Items:	Quantities, cu. yds.	Amount.	Unit cost.
76	Buffer timbers	....	18,013.32	.....
77	Crib fenders	....	22,162.40	.....
78	Backfilling	829,048	366,008.85	.4415
79	Filling center wall	217,939	109,797.82	.5038
80	Cleaning up	....	24,277.76	.....
	Total Pedro Miguel locks	....	$12,126,011.90	.....
	Miraflores east dam and spillway—			
81	Dry excavation	166,251	$ 281,455.64	$ 0.6930
82	Masonry, mass	74,221	508,151.35	6.8465
83	Masonry, machinery installation	59	3,505.78	59.4200
84	Ironwork	....	31,496.81	.....
85	Gates and caissons	....	52,911.41	.....
86	Operating machinery	....	101,805.74	.....
87	Cleaning up	....	2,110.66	.....
88	Channel south, dry excavation	4,520	1,815.27	.4016
	Total Miraflores east dam, and spillway	....	$ 983,252.66	.....
	Miraflores west dam—			
89	Dry excavation	24,459	$ 25,680.06	$ 1.0499
90	Dry filling	1,735,059	858,302.41	.2947
91	Hydraulic filling	....	59,324.09	.....
92	Masonry	4,403	21,316.80	4.8414
	Total Miraflores west dam	....	$ 964,623.36	.....
	Miraflores locks—			
93	Dry excavation, diversions	5,885	$ 2,028.98	$ 0.3448
94	Dry excavation in lock site	2,222,582	2,065,386.14	.9293
95	Dredging excavation	309,647	142,379.66	.4598
96	Hydraulic excavation	332,703	195,299.02	.5870
97	Preparing foundations—excavation	415,981	793,058.10	1.9066
98	Preparing foundations—wood piles (l. f.)	44,705	58,874.76	1.3170
99	Masonry—mass	1,479,156	7,937,895.51	5.3665
100	Masonry—machinery installation	21,102	248,449.10	11.7737
101	Iron work	....	935,647.50	.....
102	Gates	....	1,550,890.02	.....
103	Emergency dams	....	452,985.50	.....
104	Operating machinery	....	1,928,688.54	.....
105	Control house—masonry	941	49,619.29	52.7304
106	Control house—iron work and miscellaneous	....	17,378.18	.....
107	Control house—machinery installation	....	957.05	.....
108	Buffer timbers	....	7,392.86	.....
109	Crib fenders	....	51,579.96	.....
110	Backfilling	2,220,413	924,704.20	.4165
111	Filling center wall	246,655	156,432.19	.6342
112	Cleaning up	....	46,679.63	.....
	Total Miraflores locks	....	$17,566,326.19	.....
	La Boca locks and dams (abandoned)—			
113	Dry excavation	78,233	$ 158,343.29	$ 2.0240
114	Construction—dams	....	315,350.07	.....
115	Construction—locks	....	159,306.40	.....
	Total La Boca locks and dams	....	$ 632,999.76	.....
116	Naos Island breakwater	1,686,959	$ 593,918.09	$ 0.3521
	Total construction cost, Pedro Miguel to sea	....	$48,022,317.16	.....
117	Plant and equipment to be absorbed in construction cost after Dec. 31, 1913	....	828,996.51	.....
	Total expenditures, Pedro Miguel to sea	....	$48,851,313.67	.....
	Terminal facilities, Cristobal—			
118	Fuel oil tanks	....	$ 49,717.90	.....
119	Coaling plant	....	172,045.59	.....
	Total terminal facilities, Cristobal	....	$ 221,763.49	.....
	Terminal facilities, Balboa—			
120	Preliminary work	....	$ 37,178.24	.....
	Preparing site—			
121	Miscellaneous	....	$ 385,283.68	.....
122	Excavation	418,562	263,506.68	$ 0.6296
123	Filling	740,100	319,572.25	.4318
	Total preparing site	....	$ 968,362.61	.....
124	Yards and tracks	....	$ 18,130.34	.....
125	Dredging inner basin	2,663,276	447,237.25	$ 0.1679
126	Main dry dock	....	508,286.73	.....
127	Dry Dock No. 2	....	1,961.87	.....
128	Coaling plant	....	192,539.62	.....
129	Sea wall	....	82.61	.....
130	Permanent shops	....	1,251,001.17	.....
131	Docks	....	820,330.27	.....
132	Fuel oil tanks	....	36,666.63	.....
	Total construction cost, terminals, Balboa	....	$ 4,281,777.34	.....
133	Plant and equipment to be absorbed in construction costs after Dec. 31, 1913	....	273,274.02	.....
	Total expenditures, terminals at Balboa	....	$ 4,555,051.36	.....
	Miscellaneous items—			
134	Permanent town site, Balboa	....	$ 178,643.46	.....
135	Permanent town site La Boca	....	119,565.05	.....

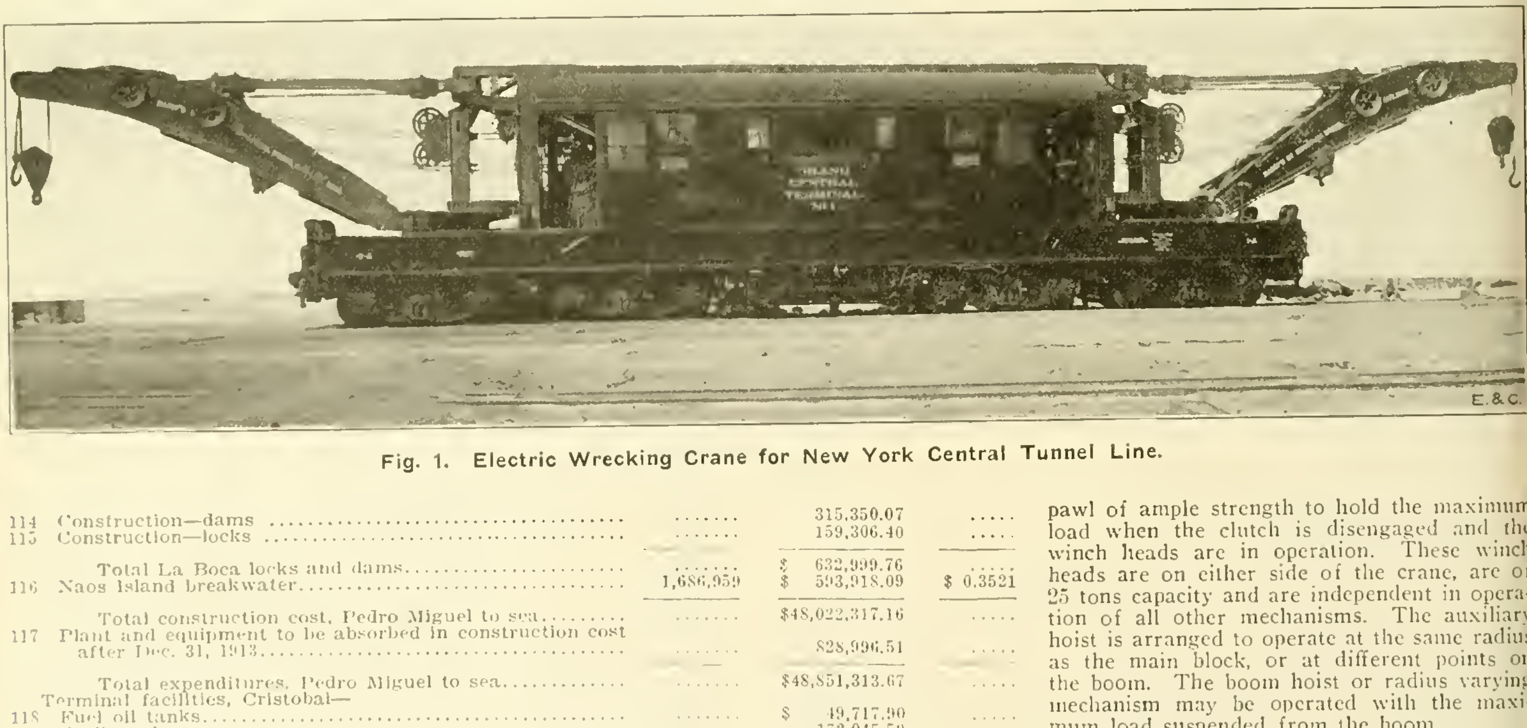


Fig. 1. Electric Wrecking Crane for New York Central Tunnel Line.

each for propelling, and two of 150 H.P. each for hoisting and for operating the machinery. The propelling motors are direct connected to the axles of the compound trucks, two at each end of the car. They are arranged so that all four may be used for traveling, or if desired only the two at either end. They are capable of operating safely on fluctuations of line voltage between 300 and 750 volts D. C. Control is from each end of the car, the controllers being arranged for connecting the motors in series, series parallel, and parallel. There is also an electrically driven air compressor.

For intermittent and emergency service, as might be required with the third rail out of commission, or when suitable cable connections could not be made with a feed line, there is installed on the crane a high capacity storage battery. This is ready for instant service to operate the crane independently of any outside source of power. It consists of 230 cells, and has a capacity of 75 amperes for 8 hours, with a maximum discharge rate of 350 amperes for 2 hours. There is a complete testing and charging outfit, in order that the battery may be charged anywhere that connection can be made with a feed line. At each end of the car is installed a switchboard suitably equipped with lamps and instruments, including an ampere-hour meter for use with the storage battery.

The operation of hoisting is by means of a train of cut spur gearing. There is a brake of sufficient capacity to hold the maximum load in any position, or to lower it at a low rate of speed. A combination clutch and brake is provided for dispatch lowering of the block. In addition there is a ratchet and pawl of ample strength to hold the maximum load when the clutch is disengaged and the winch heads are in operation. These winch heads are on either side of the crane, are of 25 tons capacity and are independent in operation of all other mechanisms. The auxiliary hoist is arranged to operate at the same radius as the main block, or at different points on the boom. The boom hoist or radius varying mechanism may be operated with the maximum load suspended from the boom.

With all four propelling motors in use the crane was guaranteed to travel at the following speeds: Hauling an 80-ton rolling load, 25 miles per hour on straight level tracks, 15 miles per hour on a 2.7% grade and 12 miles per hour on straight level tracks with the addition of a 50-ton suspended load. As a matter of fact, actual speeds greatly in excess of these were attained. At the formal test conducted by the railroad officials, the crane propelled itself at a rate of 34 miles an hour while hauling an 80-ton rolling load. Satisfactory operation is also possible, using only the two motors at either end of the car, with resulting speeds of about 60 per cent of those when all four motors are used.

In general, the following capacities are provided for: On solid foundation with outriggers; 100 tons at 24 ft. 2 ins. radius straight ahead or 6 ft. 6 ins. either side of center; 100 tons at 22 ft. radius straight ahead or 12 ft. either side of center; 100 tons at 13 ft. 8 ins.

Items:	Quantities, cu. yds.	Amount.	Unit cost.
136 Permanent town site—Pedro Miguel.	....	56,042.78	....
Total town sites	......	$ 354,251.29	
Permanent Buildings—			
137 Designing—General	....	$ 9,137.67	.....
138 Administration building, Balboa	....	169,472.14	....
139 Gold employes quarters, Balboa	....	3,887.98	.....
140 Silver employes quarters, La Boca	....	39,588.57	.....
141 Municipal buildings, Paraiso	....	883.80	..
Total, construction permanent buildings	....	$ 222,970.16	..
Power transmission system—			
142 Preliminery work	....	$ 22,768.01	...
143 Overhead transmission line	....	202,017.21	....
144 Cable direct lines	....	178,756.63	..
145 Transformer substations	....	45,800.16	..
Total power transmission system	....	$ 449,342.01	.....
146 Trans-Isthmian oil line	....	$ 34,558.73	.....
147 Lights and buoys	....	488,603.61	....
Total construction miscellaneous items	....	$ 1,549,725.80	.....
148 Plant and equipment to be absorbed in construction costs after Dec. 30, 1913	....	$ 48,671.28	.....
Total expenditures miscellaneous items	....	$ 1,598,397.08	.....
Total construction expenditures	....	$197,965,955.18	.....

Item 26 and 99—Charges to masonry in this quarter represent, to a great extent, work done in finishing, chipping, etc., and the cost has no direct relation to the yardage placed.

Item No. 57—Represents operation cement gun used in facing walls of cut.

Item No. 54—Hydraulic Excavation—This work was done on the upper banks of the Canal north of Gold Hill.

Item No. 91—Hydraulic filling Miraflores West Dam—Expenditures represent cost of pipe lines, flumes, etc. 661,048 cubic yards have been deposited in the dam from dredges and hydraulic excavation in lock chambers and in prism below the locks. The expense thereof has been charged to "Excavation."

Items 113, 114 and 145—La Boca Locks and Dams—These expenses were incurred prior to the adoption of Miraflores as the site for the lower locks and dams at the Pacific entrance of the Canal.

Item 119—Preliminary work and dredging.

Items 143 and 147—Includes charges to plant account for this work.

Items 51, 58, 117, 133 and 148—The expenditures for plant and equipment are absorbed in the construction costs on the basis of the estimated cost of plant and the estimated quantities of work to be done. The amount under these items represents the balances which are to be absorbed into construction costs of Dec. 31, 1913.

General—When excavated material is used for filling, such as dams, backfilling, etc., the cost of dumping only is charged to the item benefited, other expenses being charged to "Excavation."

Due to changes in organization, this statement no longer represents the work done by the various construction divisions, the divisions shown thereon being geographical.

swinging 180°; without outriggers 50 tons at 24 ft. 2 ins. radius straight ahead, and 25 tons at 13 ft. 8 ins. swinging 180°.

This crane was designed and built by the Industrial Works, Bay City, Mich., from specifications prepared by a committee consisting of C. H. Quereau, Superintendent Electrical Equipment (Chairman); H. A. Currie, Assistant Electrical Engineer, and B. J. Buell, Wrecking Master of the N. Y. C. & H. R. R., and was furnished through the George M. Newhall Engineering Co., eastern agents for the Industrial Works.

Automatic Street Lighting.—Experiments are being made at Leeds, England, with automatic lamp-lighting installations, and they have proved so promising that 17,000 gas lamps in that city are to be equipped. Two systems have been considered and preference has been given to the adjustable clock over the wave-pressure system. It is estimated that 77 men at 28s. 6d. ($6.93) per week can be dispensed with, which means an annual saving of about $28,000. It has been shown that failures in lights by the clock system are only 23 out of 10,000, and these can be adjusted by inspection. It is claimed that by the automatic system there is an enormous saving in mantles, and an all-around saving of nearly 20 per cent.

Portland Cement Production in 1913.—The final figures for the Portland cement industry in 1913, as obtained by the United States Geological Survey on March 16, show a production of 92,097,131 barrels, shipments of 88,689,377 barrels and stocks on hand, 11,220,328 barrels.

Drainage and Irrigation

Some Engineering Problems Connected With the Maintenance of a Water Supply Conduit.

Contributed by R. G. Hosea, Assistant Engineer, Colorado Fuel & Iron Co., Pueblo, Colo.

Many people, engineers and laymen, appear to have the idea that after a ditch has once been constructed all troubles are over and that thereafter all that has to be done is to sit on the bank and watch the water flow by. The following article will perhaps show the fallacy of this idea; it shows in some detail the troubles connected with the operation of a medium sized canal for the first seven years after its construction.

This canal is known locally as the Arkansas Valley Conduit, and takes water from the Arkansas River two miles or so below Florence, Colo. It has a capacity of about 100 sec. ft., and a water right of 70 sec. ft., and carries the major part of the water supply for the steel works of the Colorado Fuel & Iron Co. at Pueblo, Colo. This canal is approximately 38½ miles long, of which 27 2/10 miles are earth section, 4 miles are concrete lined section, 3/10 miles are concrete lined tunnel section, and 7 miles are 50 to 60 in. diameter wood pipe siphons. For about half of its length the conduit follows the river valley, almost within a stone's throw of the river itself, but gradually getting nearer and nearer to the general level of the mesas, some 100 to 200 ft. above. The river has cut down through a series of horizontal strata of limestone and sand stone, making a valley bounded by high flat topped cliffs (Fig. 1) and as the conduit follows this valley for some 20 miles, the construction of the ditch was difficult and expensive. In many places it was necessary to locate the ditch directly under the cliffs and to build concrete penstocks in dangerous places where any leakage would be sure to cause settling, if not an actual land slide.

The seven miles of siphons consist of 25 wood stave pipes, varying in diameter from 50 to 60 ins., in length from 100 to 5,300 ft.; the heads run up to 150 ft. These siphons carry the canal water across gullies or arroyos and one of them carries the ditch under the cement-manufacturing town of Portland, Colo.

Generally where the grade of the ditch was such as to bring it at the base of the cliffs, a concrete lined section was built supported on the lower side by a retaining wall, and all tunnel sections were concrete lined as indicated by Fig. 2.

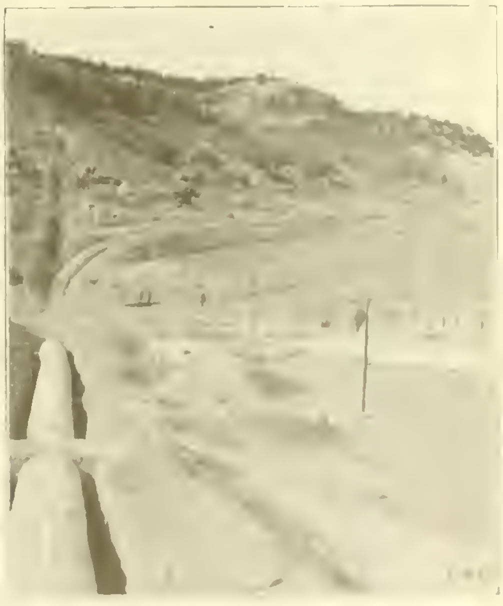


Fig. 1. Typical Country Traversed Showing Reconstruction of Siphon No. 16 Around Landslide, Arkansas Valley Conduit.

After the ditch gets out of the river valley the construction is not so difficult, being through the usual western adobe soil.

The first real trouble was encountered at the intake ends of several of the siphons, notably Nos. 11, 16 and 24. At each of these places the siphon carries the ditch across a wide, deep arroyo, and the penstock is perched upon a rocky point at the "jumping off place." The water coming down the ditch toward the penstock would apparently find a pervious stratum on the upper side, follow along it until it came to the more impervious rock, and saturate the whole hillside below this plane, causing a slide which would take with it the penstock and more or less of the ditch section. At Siphons 11 and 16 this was overcome by increasing the length of the pipe, carrying the new pipe around the slide (Figs. 1 and 3) and building a new penstock farther up the ditch, beyond where the plane of the slide intersected it. This plane could be readily seen after a slide, as the amount of the settlement in some cases was several feet. At Siphon 24 this could not be done and a deal of money was spent in trying to make the section water tight and to make the penstock "stay put," but these slides once started appear to settle indefinitely, and flumes and linings would merely pull apart. Finally a line change was made at this point (Fig. 4) involving the construction of a tunnel 300 ft. long and the taking down and rebuilding of some 1,250 ft. of pipe. The approaches to this tunnel contained 417 cu. yds. of earth, 1,897 cu. yds. of loose rock and 2,115 cu. yds. of solid rock and the concrete lining and new penstock contained 177 cu. yds. of concrete. The work on the tunnel, lining and penstock was done by Maurice Ryan of Pueblo and the pipe line change was done by the ditch crew.

These slides could not be avoided as the character of the material was necessarily treacherous, being largely debris from overhanging cliffs, composed often of rock covered with soil, resting on a sloping shale base.

In some cases slides have been stopped by carrying the water in a lined flume laid in the ground and tamped around with impervious material. When the water has drained out of the sliding material, consolidation takes place and stops the slide. These flumes are lined with waterproof tarred felt, and the seams are caulked with tar and pitch. The illustration (Fig 5) shows one about 200 ft. long over a clay slide. It is expected that the material will be solidified by the time the flume rots away.

The next difficulty encountered was in the handling of sand, of which the Arkansas River carries a very large amount. It was at first expected to handle all the coarser sand in four basins, which could be operated two at a time, the water settling in one pair while the other was being cleaned out. This method proved costly and inadequate, and the ditch and siphons below the basins began to fill up and cut down the carrying capacity. This trouble was overcome by building a sand gate (Fig. 6) one-half mile below the head gate of the ditch with a waste channel leading back to the river. The grade of this channel was 3½ ft. below the grade of the ditch at the sand gate, and it is found that by opening the sand gate for a few hours the water scours out all the sand deposited in the first half mile of ditch, thus creating a pocket one-half mile long in which new sand is deposited when the gates are closed. This sand gate is almost automatic and since its installation the trouble with sand has been reduced to almost nothing. The building of this sand gate was complicated by the fact that the waste way had to go under the main lines of both the Santa Fe and the Denver & Rio Grande railways and that in excavating for it water was encountered.

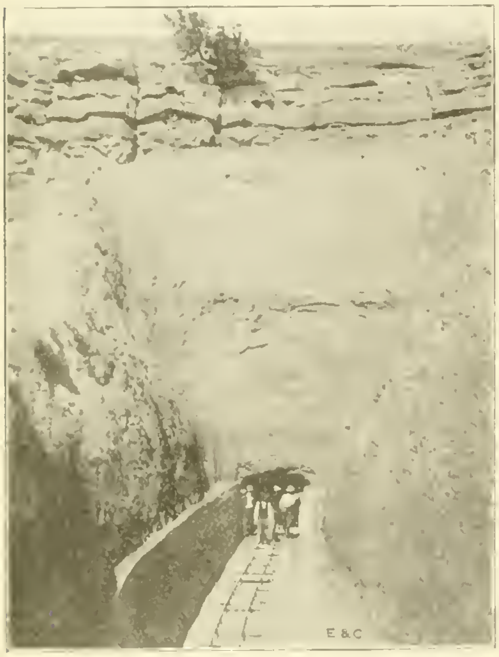


Fig. 2. Portal of Tunnel at Siphon No. 24, Arkansas Valley Conduit.

As the storage capacity of the steel works water supply is limited, being less than 1,500,000,000 gals., while the average daily use is 20,000,000, exclusive of seepage and evaporation losses, it has been necessary to run water in the ditch every possible day. In attempting to do this it was found that it was practically impossible to operate a comparatively small ditch having so many siphons, in the winter time, on account of the slush ice which would plug up in the penstocks and siphons causing endless trouble. At one gulch crossing a 50-in. pipe descends to 150 ft. below the intake and has a grade of 0.3 per cent across a length of 2,797 ft. This siphon, after carrying for a time water containing small floating ice cakes and slush or anchor ice appeared to choke up and the flow ceased. As the head above the intake increased, the obstruction would loosen and water begin to flow, but in the meantime the ditch at the outlet of the siphon would run dry and the siphon would discharge a plug 100 ft. or more in length made up of ice particles, which would fill the ditch section, so that when the water finally appeared it would run out over the banks of the ice-filled section and cause a washout. Just what is the cause of this phenomenon we have not fully determined, but we suppose that in the ascending leg of the siphon the ice and water separate, with the ice on top or ahead. Then when the flow is re-established, the ice is forced out as a plug. Whatever the cause, it is disastrous and discourages efforts to operate in very cold weather. It might be stated that the approach to this siphon is over an open mesa country for six or eight miles, where the cold at night is great, and the conditions favorable for the formation of shore and anchor ice in the comparatively shallow ditch section. In attempting to run water under such conditions a waste or turnout gate at the intake end of such siphons is a necessity to avoid wrecking the ditch.

This illustrates some of the difficulties of operating a small ditch in winter. Larger ditches which use flumes over arroyos can often carry water through the winter season by carefully regulating the flow so as to first secure an ice sheet over the surface and afterwards maintaining a uniform flow beneath the ice. This does not seem possible where the inverted siphon is a necessity, as the ice breaks, tips up on edge and dives into the intake often packing there and choking the flow or else acting as above described at the ascending part of the siphon.

After the water is turned out in the winter the ditch section fills up with snow and ice and when the water is turned in again it is necessary to follow the water foot by foot down the ditch, breaking up ice jams and nursing it along a little at a time. In this work a little home-made fire cracker, consisting of a part of a stick of giant, with cap and fuse attached, has been found very useful. One of these dropped on an ice jam does more work than three men with ice hooks.

Siphon No. 10 as originally built, was 4,673 ft. long, a 60-in. diameter wood stave pipe; 3,173 ft. of this carried the water across two gulches. The other 1,500 ft. lay directly under cliffs from 50 to 100 ft. high and directly above the Santa Fe railroad, a source of constant danger to the railway and of anxiety to all connected with the ditch. One night a part of the cliff fell over on to the pipe. A night watchman was always kept at this place, but he was at the other end of his beat, and before the break was discovered and the water turned out, a considerable land slide had gone down onto the Santa Fe track. Later a freight train ran into the slide and ditched a few cars, but fortunately injuring no one. To avoid a repetition of such accidents a change of location was necessary. The pipe was diverted up the first gulch, a distance of about 1,200 ft., and a tunnel 1,320 ft. long, through a very tough black shale was driven at some distance back from the cliffs. This came out in the same gulch originally crossed by the second part of siphon No. 10 so the old pipe line was swung over to meet the new location, a distance of 1,000 ft. or so. This work had to be completed in the winter when the water was out of the ditch. Bids were received and the contract awarded to the Phillips & O'Gara Construction Co. of Denver. The figures for the final estimate showed: Earth, 1,575 cu. yds.; loose rock, 670 cu. yds.; solid rock, 842 cu. yds.; tunnel excavation, 1,317 lin. ft., and concrete lining and penstocks, 1,058 cu. yds. The contract did not cover the changing of

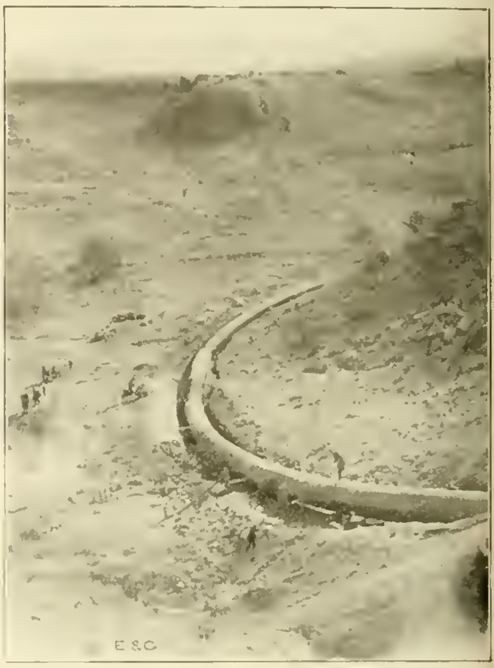


Fig. 3. Reconstruction of Siphon No. 11, 60-in. Wood Stave Pipe, on 20° Curve Around Slide, Arkansas Valley Conduit.

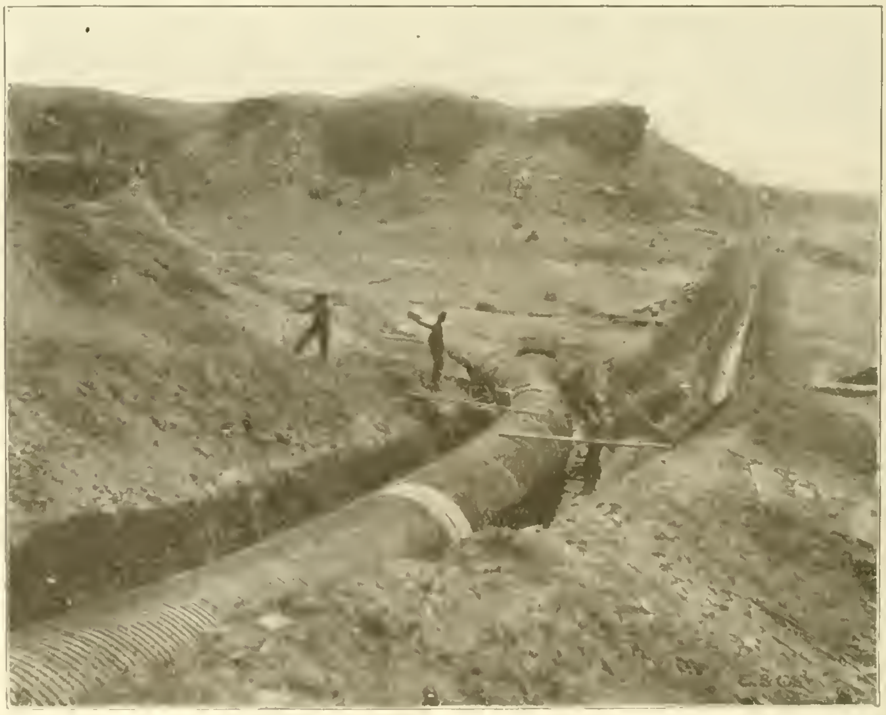


Fig. 4. Realignment of Siphon No. 24 to Meet Tunnel, Arkansas Valley Conduit.

the wooden pipe which was done by our own men. The original pipe was maintained across the slide until the tunnel was completed, when it was taken down and the old lumber used in the construction of the new pipe, saving some 1,300 ft. of pipe.

The tunnel was driven under the direction of Mr O'Gara and is one of the best jobs of its kind to be found. The section was 7 ft. wide and 7 ft. high at the arch ring, and contained approximately 2 cu. yds. excavation per lineal foot and about ¾ cu. yd. of concrete lining (about 6 ins. thick) per lineal foot. Machine drills were used, operated by compressed air, and 11 holes were drilled for each round of shots. Two shifts were worked at each face so that four rounds of shots were fired per day, using 75 to 80 lbs. of 40 per cent giant per round. The tunnel was driven at an average rate of 20 ft. per day and the two faces came together with no appreciable difference either of line or grade.

This was an expensive change, but it did away with the most dangerous place on the line and every one felt better when it was completed. The tunnel was surveyed, driven and lined with concrete in less than five months, during much of which time the weather was very bad.

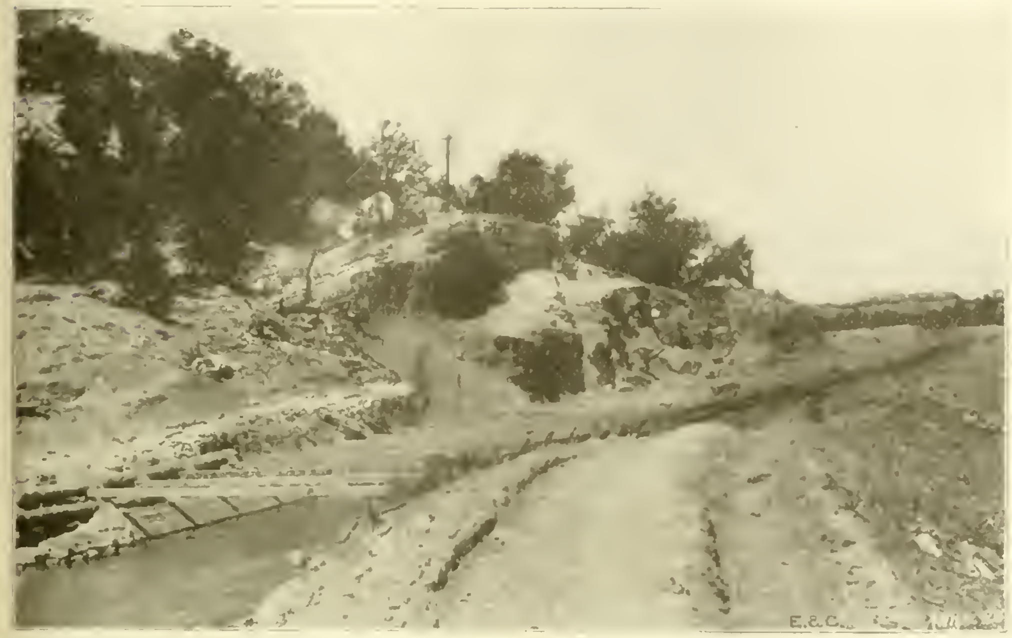


Fig. 5. Temporary Flume Built Over Slide Until Consolidation, Arkansas Valley Conduit.

Fig. 6. Sand Gate on Arkansas Valley Conduit.

As noted above, there are on the Arkansas Valley Conduit, 25 wood stave pipe siphons, aggregating 37,000 ft. These were built at an average cost of $1.96 per lineal foot, or $183,520, exclusive of trenching. In the summer of 1909 it was found that certain other pipes, which had been in use less than ten years, were so badly rotted as to be almost gone, the sound wood in many places being very thin. On this account some of the siphons on the Arkansas Valley Conduit were uncovered and inspected, and although they had been in place less than three years, some rotten wood was found showing that something had to be done to protect them and the large investment they represented. It was finally decided to enclose them in concrete as being more permanent and cheaper than anything else. The rotting of the staves in these pipes takes place on the outside in contact with the soil. As the pipes are buried in a trench with a light cover of porous soil, excellent conditions of heat and moisture are set up to encourage dry rot. Even under high pressure where the wood is saturated with water the soil absorbs it and leaves the surface subject to attack. An impervious coating of tar or paint on the outside of such pipes will produce saturation of the wood but is not permanent. The effect of dry rot is to allow the steel bands to sink into the staves and produce leaky joints. In time it is no longer possible to cinch the bands or the staves buckle and new ones must be put in. The concrete casing, put on while the pipe is under pressure, after carefully stopping all leaks, will stop the rotting of staves, since they will be assured complete saturation, and the air and possible soil influences are cut off. The preservation of bands and shoes embedded in concrete is also assured. It is necessary to backfill the pipes to prevent injury to concrete by freezing, since it will also be saturated more or less by water under pressure from within.

This work has been done by the Pueblo Bridge Co. of Pueblo, Colo., and there have been placed about 12,500 cu. yds. of concrete up to March 1, 1914. This practically completes all of the siphons on the Arkansas Valley Conduit. Figure 7 shows the work of encasing one of the siphons in progress. It will be noted that removable steel sectional forms are used and as the same forms have been used since the beginning the form expense on the whole job has been very low. The greatest cost is due to the inaccessible location, often on steep hillsides and under high cliffs where hauling of material is difficult. This concrete casing has cost about $1.33 per lineal foot of pipe, exclusive of cement and forms, and cost of digging out pipe trench. The resulting concrete pipe has been very satisfactory, except in one place where there is some very strong seepage water which is disintegrating the concrete. It is expected that the concrete casing will prevent any further decay of the wood and for that reason no reinforcing was used other than that supplied by the pipe bands.

While the sand gate mentioned above takes care of the sand in the upper end of the ditch, still a certain amount of finer sediment is constantly being deposited in the earth sections, and the heavy storms bring in a large amount wherever the ditch intercepts local drainage. The problem of removing this is troublesome. In the winter when the water is turned out, the channel is filled with snow and ice, and it would be almost impossible to clean out this frozen sand and mud at such a time. We have been experimenting with a small dredge boat which carries a gas engine and a centrifugal pump, the pump intake pipe drags along the bottom of the ditch behind a cutter or agitator, which stirs up the mud. The pump then lifts this liquid mud over the bank of the ditch. This boat has not yet been in operation long enough to tell just how efficient it may be, but it seems to do the required work very well at a low cost.

On Dec. 4, 1913, with no previous warning, a large section of hillside slid away from the adjacent cliff, taking with it some 220 ft. of concrete lined ditch. This lining, so far as we know, was water tight; at any rate it showed no signs of leakage nor did any signs of seepage show up in the level ground below the hill. The location here was such that only one thing could be done, viz: to shoot a bench out from the cliff and rebuild the ditch on it. This was rather a large job as the cliff was 52 ft. high at this point, but in time the cliff was shot down, a bench made and a concrete section reinforced with old pipe bands was built upon it. The work was done by the regular ditch crew and a few extra laborers, and Feb. [illegible], 1914, saw the water running in the new channel.

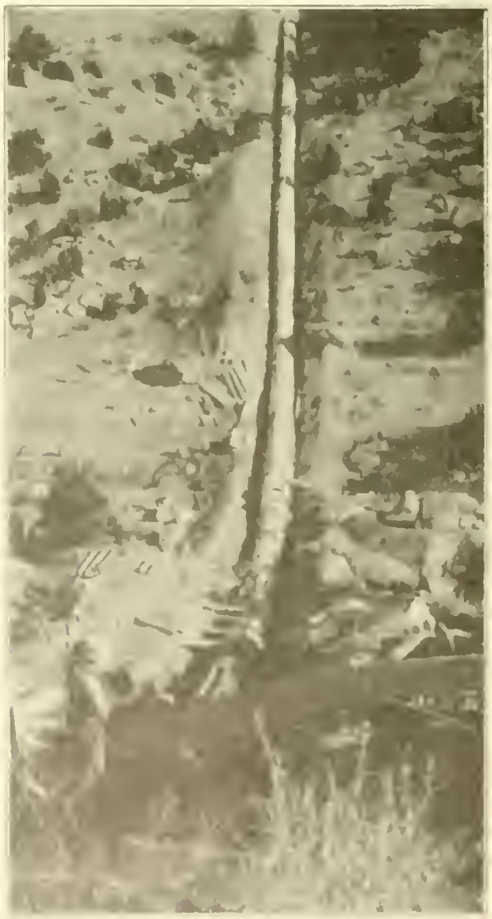


Fig. 7. Encasing Rotting Wood Stave Pipe in Concrete, Arkansas Valley Conduit.

These are a few of the larger problems such as might come to any manager or ditch superintendent, but there are a thousand others, smaller perhaps but imporant nevertheless. Water courses, which at the time of construction seemed too small to require more than a small culvert, grow from year to year, until it becomes necessary to span them with bridges or very much larger culverts. Every structure built must be extra heavy and strong and must be founded upon a rock, for water is a treacherous element to control and, like some other things, it works while you sleep. Prairie dog holes in the banks are a source of constant danger, especially in the fills.

At one place, where the ditch is carried across a shallow valley on a fill, there is an 8-ft. arch concrete culvert to take care of the local drainage. During a very heavy summer storm this culvert became blocked with a matted mass of Russian thistles and hail stones, and the water went over the ditch bank, washing out a large part of this fill. The drainage area tributary to this culvert is very small and the 8-ft. arch seemed ample for any possible flow. This cloudburst was purely local and a small drainage area caught all the water that fell and brought it quickly to the culvert, along with all the weeds and trash over the whole surface area. In replacing this fill the ditch section was lined with a board flume and tamped with impervious material, the theory being that the fill will become consolidated and firm by the time the flume rots away. It is difficult to see what means can be used in cases of this kind to guard against washouts.

The question of damages to land, cattle, etc., by reason of the operation of a ditch, though perhaps strictly a legal one, is continually coming up and involves the exercise of considerable diplomacy and tact. While the damage done by an actual break in a ditch is readily discernible, that done by seepage to subjacent lands is a constant source of dispute between the ditch company and the land owners.

The manager of a canal or ditch company lives in a pleasing state of uncertainty as to just what will happen next, and his men on the job get to be expert at carpentering, making concrete, handling powder, and doing all sorts of repair work. The variety is interesting and instructive, but the cost of a ditch and the engineering problems connected with it are by no means finished with its construction.

Diversion Dam, Spillway and Controlling Works of Universal Design for U. S. Reclamation Service.

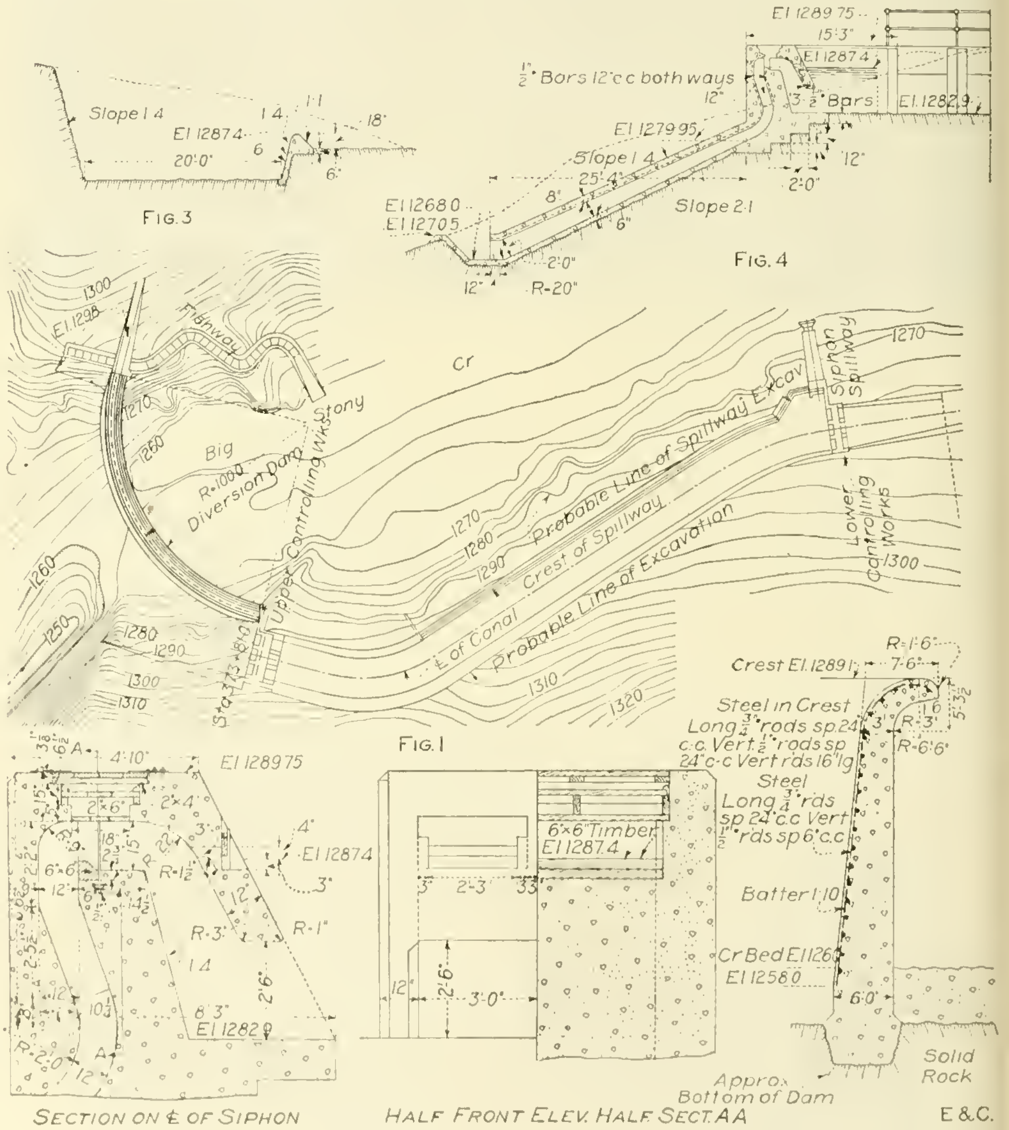
(Staff Article.)

The East Park Feed Canal is being built to divert water from Stony Creek into the East Park Reservoir of the Orland irrigation project in Calusa County, California. At Stony Creek the diversion structures shown by Fig. 1 are planned. They comprise an arch diversion dam, upper controlling works, a canal with a spillway along one edge, lower controlling works and a siphon spillway. The universal features are the dam section, the canal with an over edge spillway and the siphon spillway. Referring to the general plan in Fig. 1, the location of the dam across the narrow gorge of the creek is prominent. At the right hand end of the dam and between it and the bank are set the upper controlling works. From there a canal is excavated along the side of the bluff and extending downstream to a second set of controlling works which discharges into the feed canal proper. The river edge of the canal between the controlling works is the main spillway; the siphon spillway is just above the lower controlling works on river side.

The dam proper is an arch with a radius of 100 ft. between two abutments, one of which is a retaining wall of gravity section extending into the side of the gorge. The arch portion of the dam has a crest length of 155 ft. and at its highest point is 39.1 ft. from base to crest. The section of the dam at this maximum height is shown by Fig. 2. The notably thin section is the feature of interest and the drawing has been made to give the reinforcement and all dimensions in detail. The concrete, as required by the specifications, will be proportioned generally 1:7 to 1:9, the larger numeral representing the sum of the separate volumes of sand and gravel, but the exact proportions may from time to time be fixed by the engineer. All sand is to pass a ¼-in. screen and at least 90 per cent is to be retained by a 100-mesh screen and the particles are to be so graded that the voids shall not exceed 35 per cent. The gravel is to pass a 2-in. and to be held in a ¼-in. screen.

A section of the spillway canal is given by Fig. 3, showing it to be a plain trapezoidal channel with one side formed by a concrete weir over which the water can spill down the gorge side into the river.

The siphon spillway is shown in longitudinal section by Fig. 4. The lateral dimensions are 6-ft. width of throat decreasing to 4-ft. width at the outlet. The details of the siphon head are shown by Fig. 5. They are so complete that no further structural details are needed. The concrete for this siphon is to be the same as that used in the dam.



Map and Structures of Diversion Works, East Park Feed Canal, Orlund Project, U. S. Reclamation Service.

Fig 1—Plan of Diversion Works. Fig. 2—Section of Diversion Dam. Fig. 3—Section of Spillway Canal. Fig. 4—Longitudinal Section of Siphon Spillway. Fig. 5—Details of Siphon Head Construction.

The supervising engineer of the Orland project of which the works illustrated are a part is F. G. Hopson.

That commercial automobile lines will solve the railway problem in the Philippines is the opinion of the Governor-General, according to a recent statement. A line was recently started in Leyte, where roads are in excellent condition.

Sudan Irrigation Plan.—Interesting plans are projected for irrigation works in the Sudan, according to Consular Reports. The new dam on the Blue Nile will be located about 160 miles below Khartum; the height of this dam has not been decided, and the width will be somewhat less than that of the dam at Assuan. If begun this summer it will take about three years to complete, as work can be carried on only during the dry season. Drillings already taken show an excellent rock foundation, although there are V-shaped fissures which will have to be overcome. The canalization would be straight with few difficulties in construction. The proposed canal will irrigate 100,000 acres of land, and this acreage will in time be increased to 500,000. The scheme for the construction of a dam on the White Nile about 60 miles south of Khartum is still in prospect. Lord Kitchener is at present devoting himself to the extension of the irrigation of the Delta, three zones being first taken in hand. This work is necessarily slow and expensive, and will cost a considerable sum of money, but it will bring under cultivation several hundred thousand acres of exceptionally good land. With its completion the need for the construction of the White Nile dam, which will cost $5,000,000, will become more than ever imperative. The site for the latter is a good one, there being a rock bottom, while the river bed is wide and therefore capable of holding up a great supply of water.

Roads and Streets

Notes on Survey Practice at Storm Lake, Ia., With an Accurate Method of Balancing Earthwork from Profile.

Contributed by H. L. Browne, Asst. Eng., Buena Vista Co., Storm Lake, Ia

A good way to designate each road or section of road for filling or construction purposes is to start at the N. E. corner of the county and letter and number the sections as shown in Fig. 1. The road designated F-7-5 means the two miles of road between sections 29-30-31-32. Road 3½-C-D means a road on the half section line between 3 and 4. In designating bridges the distance from the section corner is given, i. e., Bridge F-6-7-43 means a bridge 0.43 miles from the N. W. cor. Sec. 32.

In their county two instrument men and two rodmen take the field notes for three miles of road in two days. Two parties are formed. One party chains the distance, sets stakes at 100 ft. intervals along one fence line and notes the plus distances to important points. The other party sets flags in the center of the road every 1,000 ft. to sight by and begins to take cross-sections. The first party after running well ahead takes the drainage notes and helps to finish the cross-sections.

In working up the notes, the rodman plats cross-sections and the instrument man finds the "average ground line" (explained later) for three miles of road in two days' time. It takes the instrument man three days to plat the profile and lay the grade line for three miles of work.

TABLE I—COST OF FIELD WORK IN SURVEYING THREE MILES OF ROAD IN BUENA VISTA CO., IA.

Item.	Cost.
Automobile hire, 2 days at $4.00	$ 8.00
Board, 8 dinners at $0.50	4.00
Instrument man, 11 days at $5.00	55.00
Rodman, 6 days at $2.50	15.00
Supplies—Profile paper, etc.	5.00
Total for three miles	$87.00
Cost per mile	29.00

The cross-sections shown in Fig. 2 (a) are similar to those advocated by the Iowa State Highway Commission and are in use here on 66-ft. roads.

The method of computing the excavation and balancing cut and fill which is given below has given good results for us. The results obtained have been checked by exact computation on several sections of work both on flat prairie roads and roads having as high as six per cent grades. In all cases these computations have checked within two per cent.

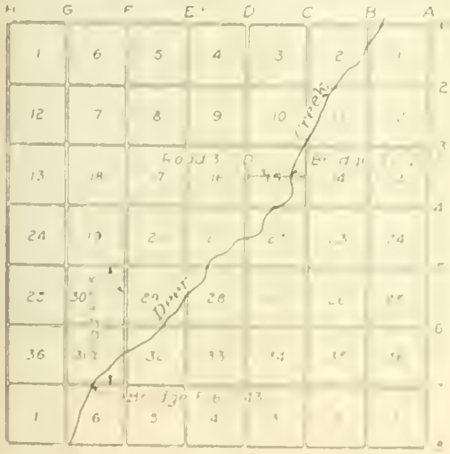


Fig. 1. Method of Designating Roads and Bridges.

The "average ground line" is defined as a horizontal line so placed on the cross-section of the existing road-bed that the sum of the areas above the line is equal to the sum of those below the line, i. e., the excavation and embankment at the section are equal. It is the average elevation of all points on the existing ground cross-section. Fig. 2 (b) shows the "average ground line" for several sections.

To locate the "average ground line" a datum is assumed (usually through the low point in the ditch) and the area between the datum and the ground line found for a width of 50 ft. This area may be computed, or found with a planimeter.

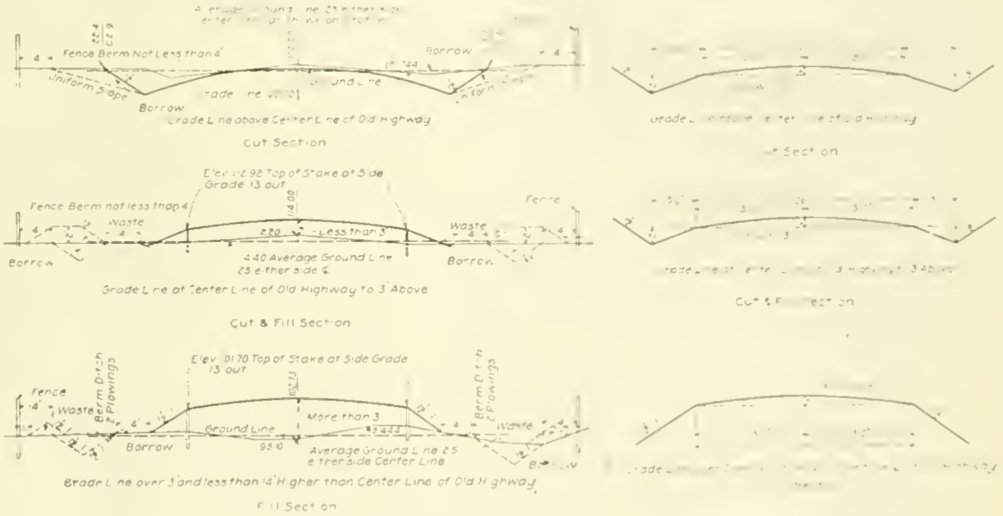

Average Ground Line 25 either side
Fence Berm Not Less than 4
Borrow
Uniform Slope
Ground Line
Grade Line
Borrow
Grade Line above Center Line of Old Highway
Cut Section
Elev. 112.92 Top of Stake at Side Grade 13 out
Fence Berm not less than 4
Waste
114.00
Less than 3
Fence
Waste
Borrow
4.40 Average Ground Line 25 either side ℄
Borrow
Grade Line at Center Line of Old Highway to 3' Above
Cut & Fill Section
Elev. 101.70 Top of Stake at Side Grade 13 out
Fence
Waste
Berm Ditch & Plowings
More than 3
Ground Line
Borrow
Average Ground Line 25 either side Center Line
Berm Ditch & Plowings
Waste
Borrow
Grade Line over 3' and less than 14' Higher than Center Line of Old Highway
Fill Section

Fig. 2. Cross Section for Roads in Buena Vista County, Iowa.

Then,

$$\frac{\text{area}}{50} + \text{elev. of datum} = \text{elev. of "av ground line."}$$

The width 50 ft. has been found by experiment to give accurate results in this section.

(The "average ground line" may be located with a fair degree of accuracy by eye after a little practice.—Editors.)

The "average ground line" need not be determined for every station but only where a marked change in the shape of the cross-section occurs.

The two sets of elevations are now plotted (preferably to a scale of 1 in. = 100 ft. horizontal and 1 in. = 1 ft. vertical), as shown in Fig 3.

In Table II is given the volume of excavation or embankment for various depths of cut and fill where the cross sections illustrated in Fig. 2 are used.

TABLE II. CUBIC YARDS PER 100 FT., COMPUTED FOR CROSS-SECTIONS ILLUSTRATED IN FIG. 2

Depth in ft.	Tenths	0.0	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9
Depth of cut in ft. 6		1,368									
5		1,111	1,162	1,187	1,208	1,230	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
4		924	[illegible]	966	988	1,009	1,031	1,053	1,071	[illegible]	1,115
3		718	738	758	779	799	820	840	861	882	903
2		524	543	562	581	600	620	639	[illegible]	679	698
1		341	358	376	394	[illegible]	[illegible]	449	468	486	[illegible]
0	Cut	Grade.	186	202	219	[illegible]	[illegible]	270	288	[illegible]	323
Depth of fill in ft.	Net cut	171	150	131	117	109	[illegible]	68	[illegible]	36	20
0	Cut	171	153	140	127	116	[illegible]	97	88	81	73
	Fill	0	[illegible]	6	10	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
	Net fill	[illegible]	11	26	[illegible]	[illegible]	[illegible]	87	101	[illegible]	130
1	Cut	66	[illegible]	[illegible]	48	[illegible]	[illegible]	[illegible]	29	[illegible]	21
	Fill	61	70	80	90	[illegible]	110	[illegible]	130	140	[illegible]
	Net fill	[illegible]	[illegible]	171	[illegible]	[illegible]	211	[illegible]	236	[illegible]	[illegible]
2	Cut	18	[illegible]	12	9	7	5	4	2	1	1
	Fill	161	172	183	194	[illegible]	216	227	238	250	261
3	Fill	273	[illegible]	297	309	[illegible]	[illegible]	[illegible]	358	[illegible]	[illegible]
4		[illegible]	409	422	[illegible]	[illegible]	462	477	489	[illegible]	516
5		[illegible]	544	[illegible]	[illegible]	[illegible]	601	616	631	645	660
6		[illegible]									.....

Fig. 4 illustrates a device for the rapid taking off of quantities from the profile. It is made of a piece of "Plate A" profile paper 2¼ ins. wide with a ¼-in slot cut in the center. Along this slot are written the net volumes of excavation and embankment corresponding to a cut or fill below or above the "average ground line" taken from Table II. By net cut or fill is meant the excess of cut or fill at the section beyond that required to make the cross-section at that point, quantities being computed from a level section.

An approximate grade line is now laid. This should meet all controlling features such as R. R. crossings, bridges, maximum grades, etc., and should be so placed that the excavation and embankment will balance as nearly as it is possible to judge with the eye. To rapidly determine whether the cuts and fills will balance the device illustrated in Fig 4 may be used.

The scale is applied to the profile, the point marked "grade" being placed on the proposed grade line. The net excavation or embankment for any cut or fill between the proposed grade line and the existing "average grade line" is read directly from the scale for any point on the profile. The "average grade point" is defined as the point at which the excavation from the ditches makes the fill for the crown. This point is located with the scale and is found at that point on the profile where the zero cut and fill mark on the scale coincides with the "average ground line." The location of their point gives the length of the earth wedges used in computing quantities.

Example. What is the approximate excavation and embankment between Sta. 167 and Sta. 172?

Apply the scale to the profile with the point marked "grade" on proposed finished grades at Sta 167. On the scale read 85 cu. yds. cut where the "average ground line" is intercepted. Apply the scale at Sta. 167 + 50 and read zero cut and fill, and so on throughout. Make up a table similar to Table III.

TABLE III—APPROXIMATE QUANTITIES BETWEEN STA. 167 AND STA. 172 FOR BALANCING PROFILE.

	Yardage at ends above that required to make the section.		Net yardage above that required to make the section.	
Sta.	Cut.	Fill.	Cut.	Fill
167	117	0	..	..
	$\left(\frac{117}{4}\right)$		29	0
+50	0	0	..	..
168	0	0	0	0
169	0	0	0	0
170	0	0	0	0
		$\left(\frac{11}{2}\right)$		5
171	0	26	.	.
		$\left(\frac{98}{2}\right)$		49
172	0	72	.	..

By making up a table of this kind the amount of earth which must be moved some distance is determined rapidly and accurately and with the aid of this table the final grade line may be so placed that an economical disposition of the earth can be made. Only the information which is needed is given.

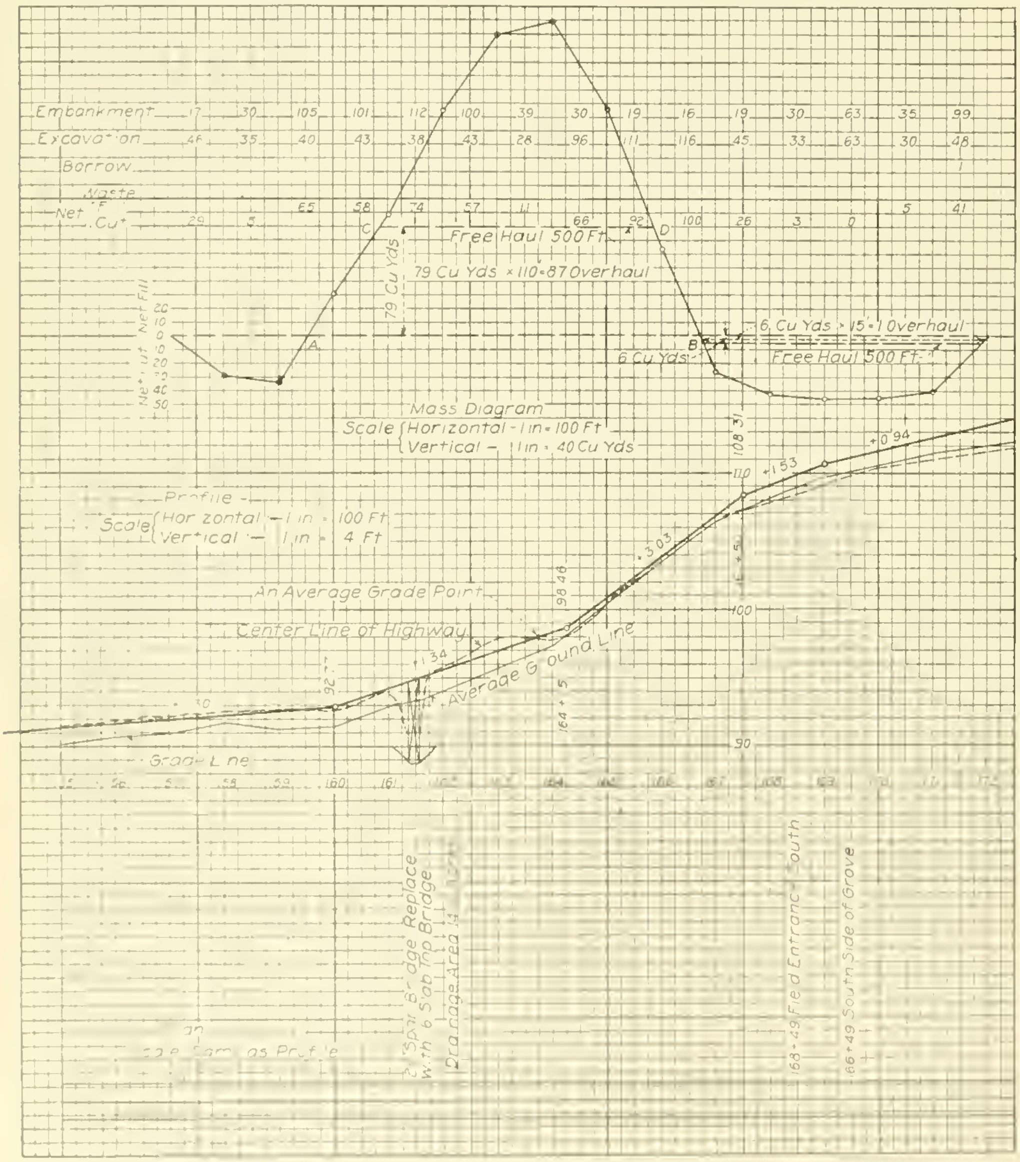


Fig. 3. Portion of Plans for Use in Road Construction in Buena Vista County, Iowa.

To obtain the final quantities to be used for final payment and for use in constructing a mass diagram to determine overhaul, a tabulation similar to Table IV (which is a table for a different section of road from that shown in Fig. 2) should be made.

In making up this table the quantities corresponding to the exact cut or fill at the station are taken from Table II.

The mass diagram for determining overhaul and the disposition of excavated material shown in Fig. 3 is constructed according to the methods outlined in books on earthwork.

Only the material to be moved beyond the section is included in it. Net yardage is plotted vertically as shown—cut down and fill up—fill being corrected for shrinkage. The balancing line is drawn. Where the material does not balance within the free haul limits (in this case 500 ft.) as at A-B, the line C-D is drawn at the place where its length between the points where it intersects the mass diagram is equal to the free haul.

$$\text{Then, } \frac{AB + CD}{2} - \text{Free haul} = \text{Overhaul distance.}$$

The yardage of overhaul is represented by the distance between lines A B and C D (in this case 79 cu. yds.) and is multiplied by the overhaul distance to obtain the yardage for compensation.

Then,

Overhaul × Compensation per 100 ft. = Total compensation.

Full information as to excavation, embankment, borrow, waste, free haul, overhaul and pay quantities of excavation and overhaul should be placed on the plans.

Roadwork in Nova Scotia in 1914.—It is estimated that about $600,000 will be expended this year for road work in the Province of Nova Scotia, Can. A portion of this total is in the form of statute labor and the remainder is contributed by the Government out of its revenues and expended on the roads in the various counties, the work being done under the direction of county inspectors. In addition an act passed last year provides for the expenditure of $10,000 in Ferris county. Funds amounting to $13,000 yearly from license fees for motor vehicles also are available for road betterment purposes. Hiram Donkin, Halifax, N. S., is Road Commissioner of Nova Scotia.

TABLE IV—FINAL QUANTITIES FOR PAYMENT AND USE IN MASS DIAGRAM.

		Yardage at ends.			Net yardage for use in mass diagram.	
Sta.	Excavation.	Cut.	Fill.	Embankment.	Cut.	Fill.
257	..	51	70	..	..	..
	3	..	..	3	..	..
+10	..	63	63	..	..	..
	72	..	..	41	31	..
258	..	97	29	..	..	..
	44	..	..	25	..	..
+55	..	63	63	..	..	..
	23	..	..	37	5	..
259	..	43	100	..	..	..
	40	.	..	105	..	65
260	..	38	110	..	..	..
	43	..	..	101	..	58
261	..	48	92	..	..	..
	25	..	..	66	..	..
+62	..	33	120	..	..	74
	13	..	..	46	..	..
262	..	33	120	..	..	..
	43	..	..	100	..	57
263	..	54	80	..	..	..
	50	..	..	61	..	..
+85	..	63	63	..	..	..
	10	..	..	9	..	10
264	..	66	61	..	..	.
	20	..	..	11	..	..
+25	..	97	29	..	66	..
	76	..	..	19	..	.
265	..	107	22	..	..	..
	111	..	..	19	92	..
266	..	116	16	..	..	..
	116	..	..	16	100	..
267	..	116	16	..	..	..
	29	..	..	4	..	..

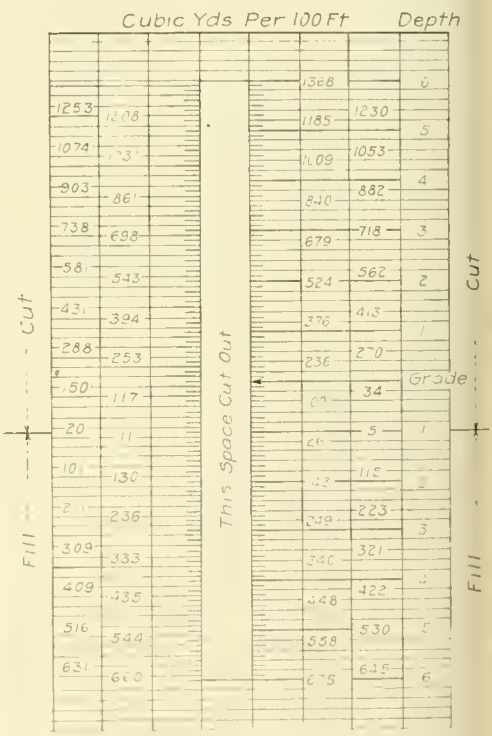


Fig. 4. Scale of Net Quantities per 100 Ft. for Use in Balancing Profile.

Curves on Brick Roads: Super-elevation.—The following table shows the super-elevation on curves used in brick road construction in Ohio. The change in level is made by depressing one side of the road and elevating the other the amount shown.

Degree of curve.	Difference in elevation of curbs, in ft.	Outside curb raised, inside curb lowered, in ft.
0—6	0	0
7	0.36	0.18
8	0.40	0.20
9	0.46	0.23
10	0.52	0.26
11	0.56	0.28
12	0.62	0.31
13 and over	0.66	0.33

All curves are to have normal elevation at P.C. and P. T. and are to acquire the maximum amount of elevation or depression at a distance of 50 ft. from the point of tangency except in special cases.

Methods and Cost of Snow Removal in Philadelphia in 1914.

Philadelphia has a widely scattered business section compared to other cities and the problem of snow removal involves the handling of large quantities of snow. The method of fighting the snowstorm, or beginning the work of removal before any considerable accumulation takes place, which is in vogue, is no doubt responsible in a large measure for the success of this city in coping with its snow problem.

BUREAU OF HIGHWAYS, PHILADELPHIA
LOADING TICKET STUB — District 11 — Ticket No. 72505
SNOW REMOVAL
ONE DOUBLE-TEAM LOAD
Exchanged for Dumping Ticket No. ....

Bureau of Highways
Department of Public Works
Philadelphia
LOADING TICKET
SNOW REMOVAL
District 11 — Ticket No. 72505

To INSPECTOR AT DUMPING STATION:
Allow bearer to dump at proper Station ONE DOUBLE-TEAM LOAD OF SNOW, and Exchange this ticket for Dumping Ticket No. ....

.... Inspector

(a)

BUREAU OF HIGHWAYS, PHILADELPHIA
DUMPING TICKET STUB — District 11 — Ticket No. 72042
SNOW REMOVAL
ONE DOUBLE-TEAM LOAD
Exchanged for Loading Ticket No. ....

Bureau of Highways
Department of Public Works
Philadelphia
DUMPING TICKET
SNOW REMOVAL
District 11 — Ticket No. 72042

THIS CERTIFIES That bearer dumped ONE DOUBLE-TEAM LOAD OF SNOW at proper Station, and this ticket was exchanged for Loading Ticket No ....

.... Inspector

(b)

Fig. 1. Forms of Loading and Dumping Tickets Used in Philadelphia, Pa.

It is also quite probable that the increased use of rapid-moving motor vehicles has had an important bearing upon the lack of customary traffic congestion noticeable in this and other cities during the past winter.

The information given in this article was prepared by Mr. Wm. H. Connell, chief of the bureau of highways and street cleaning of Philadelphia, and presented at the Philadelphia Snow Removal Conference.

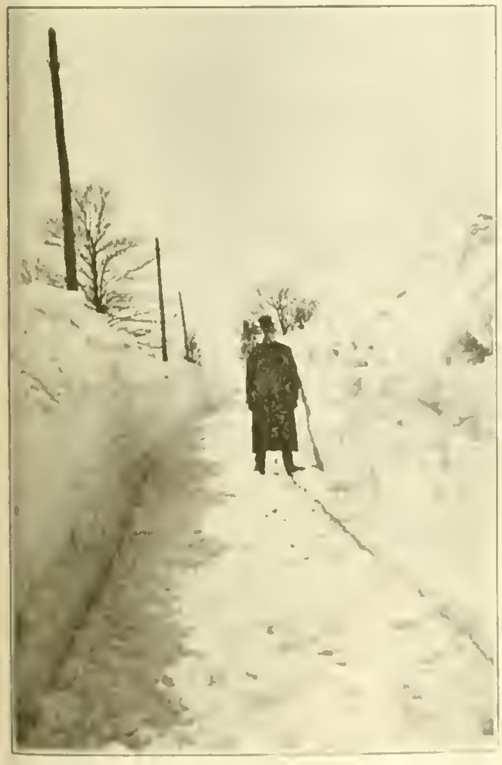


Fig. 2. View Showing the Depth of Snow on a Country Road.

Preliminary Organization.—The system employed consisted of outlining a plan of campaign designed to cope with the conditions that might arise. The engineers and inspectors were instructed and understood what was expected of them in case of a storm. As soon as the snow was of a depth of about 2 ins. the machinery was put to work to fight the storm. The disposal points were all planned and arranged for in advance, and in most cases consisted of sewer manholes, and were so located that the hauls were comparatively short.

Prior to 1912, no attempt was ever made to handle the snow in a systematic manner through the use of plows and scrapers and piling before loading, the consequence being that traffic was more or less tied up until the snow was entirely removed from the streets. The plan under which we operated this season provided that the snow be first plowed to the side of the cartway and then shoveled into piles, from which it was loaded into wagons and hauled to the disposal places. Formerly the contract for the removal of snow in the central section of the city was awarded to one contractor, with the result that the city was at the mercy of this contractor, who, in turn, was at the mercy of the smaller contractors from whom he had to secure teams for the work, as no one contractor in the city had sufficient teams available to handle the work in its entirety.

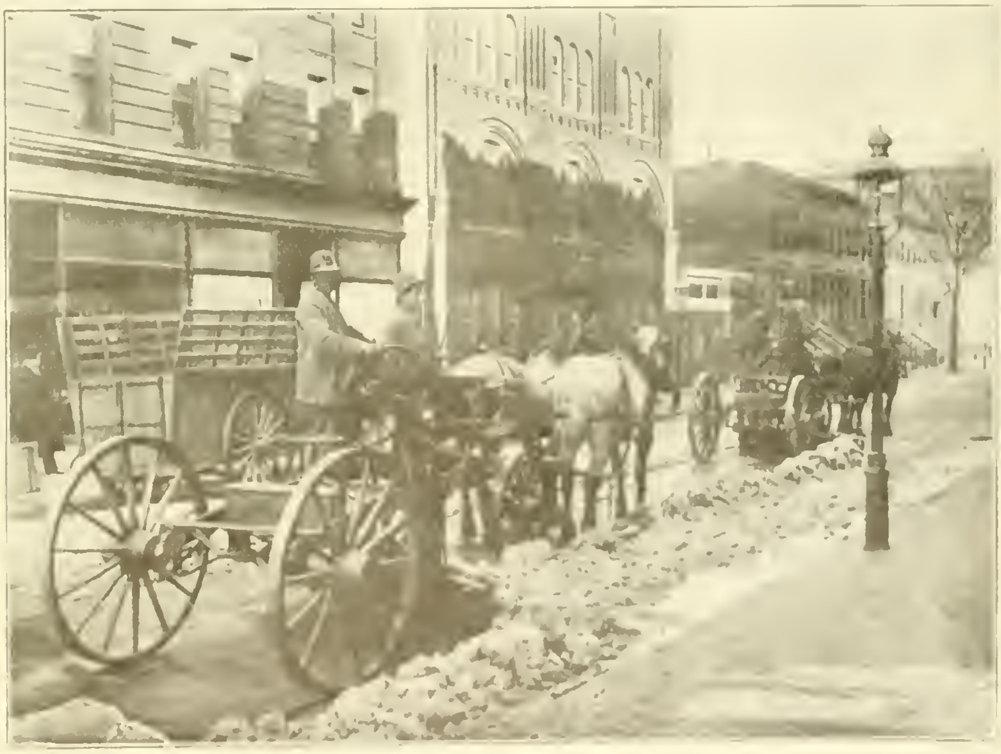


Fig. 4. Scraper Piling Snow in Street Gutter.

It was, therefore, decided that much better results could be obtained by dividing the central section of the city into 13 districts, and in advertising for bids, provision was made that no one contractor would be awarded more than two of these districts. This gave the small contractor with a few teams an equal opportunity to bid against the larger contractors, and greater competition was obtained. Furthermore, this method made it possible to start work simultaneously in the 13 different districts. The bids received ranged from 78 to 89 cts. per load of 2 cubic yards, depending upon the location of the district and the length of haul to the disposal place.

The specifications required the contractors to start two snow plows in each district within two hours after notification, and to remove a minimum number of loads of snow within 24 hours after being notified to begin work, to insure the removal of a depth of 6 ins. in this period of time.

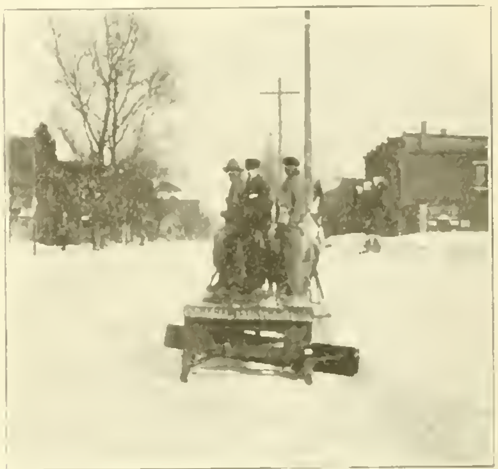


Fig. 3. Snow Scraper Clearing Center of Street.

Force Employed.—The total force at work cleaning the snow was in the neighborhood of 2,400 men, 1,000 horses and 500 wagons. Three division and eight district engineers, and 100 inspectors were engaged in the supervision of the work. One inspector was assigned to each snow district to act as a squad leader, and to each of these leaders was assigned a definite number of inspectors [illegible] first and second [illegible] with the understanding that in case of snow the squad leader was to get in touch with his inspectors and also the contractor for his district, and then communicate with the main office for orders to start work.

Method of Conducting Work.—As soon as the snow fall was of sufficient depth to warrant proceeding with the work, the contractors were ordered to start snow plows in their respective districts. The snow was plowed to the sides of the streets, after which the teams and laborers were ordered to proceed with the work of removal. In this way we managed to keep ahead of the storm on all the main business thoroughfares and succeeded in having the streets open to traffic when the snow ceased falling: this, of course, necessitated working day and night. The entire engineering force worked in day and night shifts, and were instructed to follow the plows, and under no condition to permit them to stop working until the snow was plowed to the sides of the streets. Our experience has led us to believe it was only through this method of fighting the snow that we managed to cope with the situation.

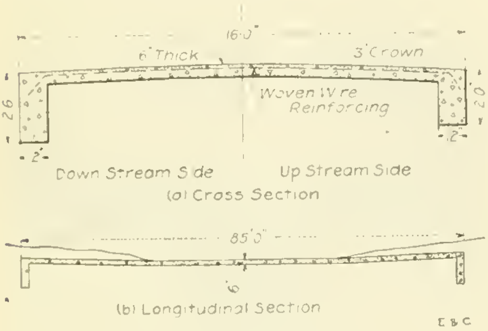


Fig. 1. Sections Showing Construction of Ford at Cowles, Nebr.

System of Checking Loads.—The following system was employed for checking the number of loads hauled: Inspectors were stationed at both the loading and dumping places. When a wagon was loaded with the required two cubic yards capacity, a loading ticket, Fig. 1 (a) properly signed by the inspector, was given to the driver; and upon delivery and dumping of the load at the dump designated for the respective district, the loading ticket was exchanged by the dump inspector for a dump ticket, Fig. 1 (b) signed by the inspector and showing the number of the loading ticket for which it was exchanged. The stubs in both the loading and dumping ticket books were properly filled in to serve as a double check.

Amount and Cost of Work.—During the months of February and March, 1914, five snow storms laid a total of 31 ins. of snow. The area cleaned in the central business section amounted to 20 miles of streets, or 851,593 sq. yds. of surface. The snow was removed moved from this area at a cost to the city of $2,275 per mile for the entire winter; the average cost per storm for the four largest storms, being $568 per mile. This, of course, does not include the work done by the regular street cleaning forces outside of the central business section of the city (all the street crossings throughout the entire city were cleaned and portions of a number of the main thoroughfares), nor does it include the force employed in opening up the country roads.

Suggested Improvements.—It is unfortunate that the city does not own scrapers and plows, as this portion of the work at least should be done by the city forces. Sufficient scrapers and plows should be owned by the city and stationed at advantageous points so that they could be put to work as soon as a storm is making sufficient headway to warrant it. In this way the city with its own forces could plow the snow to the sides of the streets, which would keep them open, and the snow could then be hauled away without seriously inconveniencing traffic.

The Philadelphia Rapid Transit Co. had its plows working all night and managed in this way to keep ahead of the storm to such an extent that the traffic was not seriously tied up. As a matter of fact, it was only during the last storm that some of the lines were out of service for a few hours, and these were in the outlying districts. In connection with the work of the railway companies, it would seem to be practical to have some sort of a plow designed that would not only clean the snow from between the tracks but for an area of eight to ten feet on each side of the tracks. This work could be carried on in the same way that we used our plows and scrapers this winter, namely, by putting the plows to work in the early stages of the storm and fighting the snow. It would pay, and would be a big help toward solving the snow removal problem on railway streets.

Most of the snow removed from the central portion of the city was dumped into sewer manholes. Unfortunately all these sewer manholes could not be utilized because of the small flow of water in the sewers. A study is now being made with a view to having water connections placed inside of all the manholes to facilitate the flow, and thus enable us to use about 50 per cent more manholes than are now available.

Design and Cost of Construction of a Concrete Paved Ford at Cowles, Nebr.

(Staff Article.)

In flat and semi-arid sections of the country where rainfalls are infrequent but usually torrential the provision of waterways of sufficient area to care for the runoff and prevent washouts is a serious problem. A possible solution for some localities may be secured in the use of a paved ford, or dip in the longitudinal profile of the road, similar to that used at Cowles, Nebr.

While a bridge or culvert large enough to provide for the water is undoubtedly more satisfactory and better road design than the paved ford type of construction, there may be conditions under which the use of a ford, or dip, is economical. This is especially true where the surrounding country is flat and in sections where the dry stream beds are sandy or composed of soils which are easily eroded.

The concrete ford built at Cowles, Nebr., is 85 ft. long and 16 ft. wide. Aggregate for the concrete was obtained near the ford. The concrete was mixed in a power mixer. The work was done by contract at the prices given below. The cost data given in Table I and the sketches shown herewith as Figs. 1 (a) and 1 (b) were contributed by Mr F. A. Good of Cowles, Nebr.

TABLE I — CONTRACT COST OF CONSTRUCTING A CONCRETE FORD AT COWLES, NEBR.

Item.	Cost
Concrete, 32 cu. yds. at $5.50	$175.00
Triangular mesh reinforcing	16.00
Total cost	$191.00
Cost per square yard	$1.26
Cost per linear foot of ford	2.25

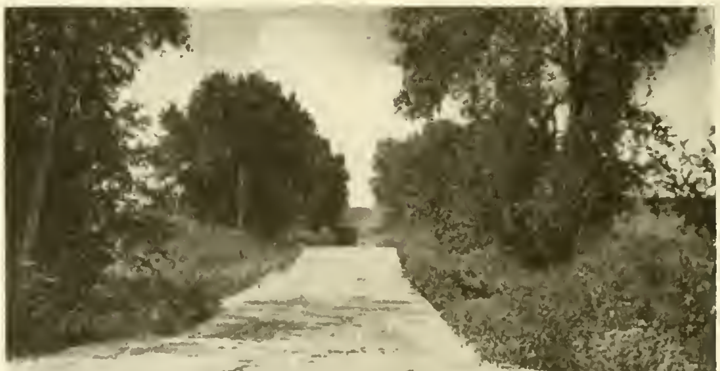


Fig. 2. View of Ford Looking Along Road.

Macadam Road Construction Without the Use of a Heavy Roller.

It often happens in improving short, isolated sections of road that a heavy roller is not available. Where a soft limestone is used in the construction it is possible to build a very satisfactory road without the use of a roller. Many of the noted Kentucky turnpikes were built in this way. These instructions were presented in the fourth report of the Illinois Highway Commission as follows:

Construction.—The essential features of such construction are a well drained and well shaped foundation to place the stone upon, the uniform spreading of the crushed stone, the proper width and alignment of the earth shoulders, which hold the bed of stone in place and the proper drainage of the road bed by means of lateral drains filled with stone.

The critical time in the life of such a road is during the first year or two after it is built, particularly the first year. After the crushed stone has been placed and covered with screenings and the road opened for traffic, it is inevitable that ruts will form along the wheel track, and if these ruts are allowed to remain in the surface of the road they will hold water and cause its rapid deterioration. If however, the ruts which form when the road is first put into service are filled with stone and the surface kept smooth by means of a drag or road grader, the stone will finally compact to form a smooth, even, durable surface.

Instead of the screenings to fill voids, a good bonding gravel is very much better and even if screenings are used, the surface of the road should be covered to a depth of about 1 in. with a good quality of gravel. This gravel, however, should not contain too great a percentage of clay or loam, because if there is a large quantity of clay in the gravel, it will become sticky during wet weather and cause the stone to pull out of the surface of the road on the wheels.

Emphasis is placed on the fact that if the surface of the road is to become fit for traffic, it should be repeatedly smoothed during the first year of service, and after that time it will require comparatively little attention. A small amount of repair work each season on the various roads will prevent any serious deterioration in them, and will maintain them in a satisfactory condition indefinitely. The best material to keep the surface in good condition is a good grade of bonding gravel which should not run over 1 in. in size. It should be spread very thin upon the road where it shows wear. Repairs of this nature should be made early in the spring so as to leave the surface well bonded before hot, dry weather. Too much emphasis, however, can not be put on the necessity for giving the roads due attention from year to year if they are to remain in satisfactory condition.

Engineering and Contracting

**Devoted to the Economics of Civil Engineering Design
and to Methods and Cost of Construction**

Volume XLI. CHICAGO, ILL., MAY 27, 1914. Number 21.

The Faulty Nature of American Water Consumption Statistics.

In his paper before the 1912 convention of the American Water Works Association Mr. Edward S. Cole called attention to the desirability of separating domestic consumption from other forms of water consumpion in reporting the amount of water used by a city. Unless such a separation is made water consumption statistics are very likely to be misleading and may have but little value for comparative purposes, although so freely used in that manner. It has always been the custom in this country to report the total consumption. Thus the average daily pumpage of water in gallons for a given city is divided by the total population of the city, or by the number of water takers if other supplies than the public one are utilized, and the quotient obtained is reported as the average daily per capita consumption of water in gallons per person. Thus, as Mr. Cole states, we lump together indiscriminately public use, business use and domestic use and compare the figures so obtained with similar figures from other cities. Moreover, the American figures, obtained as stated, frequently are compared with the consumption statistics reported by English cities, although, as a rule, the latter figures are for domestic consumption only. Many of our most prominent writers on water works subjects have fallen into the error of comparing the total per capita consumption of water in American cities with the domestic consumption in English cities while under the impression that reported figures on the use of water by the cities of the two countries are truly comparable. A mistake of a somewhat similar nature is made when the consumption statistics for a manufacturing city are compared with those from a residential city. This is clearly apparent when we consider that in many cities enough water is used for business purposes to give a per capita consumption for the whole city of over 100 gals. Many who are interested in water supply matters, particularly city officials, have failed to grasp this phase of the individuality of a water works system just as they have failed to take into consideration local influences on the cost of supplying water and the resulting effect upon water rates.

In an article on water consumption in Indiana, published in our issue of May 13, 1914, a contributor stated that it is surprising when questioning many water works operators to find how little they know about the amount of water used in their cities. This condition is said to exist in many privately owned plants as well as in the municipal plants. Many times when consumption data are given out they are based on only roughly approximate estimates and, in some cases, appear to be little more than fairly shrewd guesses. Very often the consumption data given out are in error by as much as 20 per cent owing to pump slippage which has not been properly taken into consideration. Again, water used in condensers and pumped twice is all reported as used, no allowance being made for double pumpage. For many other reasons the water consumption data given out by American cities are faulty.

Following the reading of the paper mentioned Mr. Cole was appointed chairman of a committee on water consumption and the committee has been working for some time to gather consumption statistics in which the domestic consumption of water is separated from the use of water for other purposes. The committee made a progress report at the recent convention of the Association and there is still time to report to the chairman additional data on the domestic consumption of water. Data on domestic consumption will be valuable to every water works operator and will enable him to test the efficiency of his plant in this important particular. Having in mind that only the domestic consumption of water is truly comparable, as between various cities of similar character of population, we urge any reader who may have such data to communicate them to Mr. Cole at 220 Broadway, New York. The researches of the committee will be published, it is believed, in an early issue of the quarterly journal of the Association.

Our readers in the water works field are urged to keep their water consumption statistics, so far as practicable, in such form that the use of water for public purposes, for business purposes and for domestic purposes can be reported with a degree of accuracy worthy of the importance of this matter.

The Testing of Aggregate in Concrete Road Construction.

It is unnecessary to emphasize to structural engineers the importance of dense concrete and hard, tough aggregate therefor. This requirement, however, has not been so long taught to road builders although its complete acceptance is of greater importance to them. Proportioning concrete is beautifully simple in theory—in practice it is often not so simple. The construction of a first-class concrete wearing surface demands an absolute uniformity of concrete: in hardness, in mixture, in percentage of moisture, in curing, and possibly in the temperature at which laid. Every one of the factors mentioned is controllable and subject to the improvements in methods which will undoubtedly follow the careful study being given to concrete roads by engineers at the present time.

The properties of concrete depend fundamentally upon the aggregate from which it is produced and therefore the quality of the aggregate is all important. With this in mind, recent specifications for concrete road surfaces are much more specific in this respect than formerly For instance, the requirement of a French coefficient of wear of at least twelve in the stone, the testing of sand for silica content, the requirement of carefully graded sizes of stone, are matters of importance. The quality of the concrete which comes out of the mixer is entirely dependent upon the materials which go into it. Mixing concrete is not a magic process by which poor materials may be transformed in some inexplicable manner into imperishable stone.

In this connection, the method of proportioning gravel concrete which is given in this issue of Engineering and Contracting has much to commend it. While we do not believe it is desirable to have an excess of mortar in concrete for use in a road surface, as provided for in some of the curves in this article, the systematic and continuous sampling and testing of the aggregate, which the method and outfit used provides, is of great importance in securing uniform results. The methods given were developed by Mr. A. N Johnson, of the Illinois Highway Commission for use in securing good results in the concrete bridge and culvert work under his supervision and have been successfully used, as his bridges testify. In concrete road construction this method should be used to supplement the regular laboratory tests; it is not proposed as a substitute

There are a number of factors entering into the question of proportions and not the least of these is the water content of the mixture. With the great variation in soil composition of the sub-grade upon which the surface is laid there must exist an accompanying difference in seepage and absorption of moisture from the concrete in curing. It would seem then that even should uniformity of materials and mixture be secured, the cured concrete surface would be by no means uniform. This is undoubtedly a difficult condition to overcome. Various remedies have been suggested, among them may be mentioned, the addition of a small amount of hydrated lime to the cement, the use of a relatively dry concrete, the incorporation of a small amount of clay in a sandy sub-grade. It may be stated that the final evolution of the concrete road has by no means been reached. It is possible that it may never become standardized and details of construction must be varied radically to successfully meet local conditions.

Modern Tendencies in Equipping and Operating Structural Shops.

It is essential that the designer, the detailer and the builder thoroughly understand the character of the equipment and the procedure followed in structural steel shops. Unless one is closely connected with the operation of such shops it is difficult for him to appreciate the constant changes in equipment and methods. As the writer has recently visited a number of the leading structural shops in this country and Canada and has had considerable experience in such shops in the past he will record some of the impressions created.

The greatest progress has been made in the methods and equipment for handling materials, as it is now realized that decreasing the number of operations required to fabricate a member is extremely important. The manner of storing, loading and unloading materials, and the transfer of them to and from the machines are even more important factors than the speeds of the various machines, although the use of high-speed tools has increased considerably. One marked change has been the development of heavy-duty machines for massive work. Shears, bending rolls, straighteners, saws and punches now have much greater capacities. In connection with multiple punches one of the most interesting developments has been the automatic spacing tables in use in some shops. Coincident with the use of heavier machines there is a marked tendency to separate the facilities for light and heavy work. Although this requires a greater floor space, and thus increases the overhead expenses, the increased production has resulted in lower unit costs. The operation of the large-capacity machines for light work is a very wasteful procedure.

Especially in the smaller shops, the use of combination machines, such as machines for shearing and punching, for sawing and rotary planing, for bending and straightening, etc., has greatly increased The small combination shearing and punching machines are extremely useful as they can also be taken into the field and used for such emergency service as is unavoidable in erection work. The sawing and rotary planing machines are in some shops displacing friction saws, as the planer face can be used for a number of operations.

Radial extension drills, operating from long arms which may be attached either to the walls or may be swung completely around a column, have come into extensive use. Heavy pneumatic riveters, carried by a crane which is in turn mounted on a truck, are in use in some shops. The truck runs on a track, and is operated by means of an electric motor. The increasing use of tough alloy steels for the rivet sets has reduced the breakage caused by the punching of thick plates. The increase in the use of large horizontal milling machines is marked. Several of these machines have a travel of from 30 to 40 ft., the plates being held tightly in position by pistons operated from pneumatic cylinders. Rotary planing machines are growing in favor, and duplex adjustable planers are now arranged for planing both ends of long members at the same time. An interesting development has been the milling machines, which have recently come into use for finishing the ends of braces and stiffeners and for chamfering the corners of angle braces. One of these tools is so constructed that it will mill, simultaneously, both ends of two angles. Machines for boring pin holes at both ends of long compression members are also in use, and have been developed to a high degree of efficiency.

Electric drive has now come into almost universal use, and with it there has come more complete pneumatic systems. Most of the larger machines are operated by separate motors, but it is quite common practice to connect several of the smaller machines by shafting, and to operate them from a single motor. Locomotive cranes are being used more and more for yard service and to transport the material to and from storage. The use of magnets suspended from traveling cranes or from the booms of locomotive cranes has effected economy in some plants. Although traveling electric cranes, which span the bays of structural shops, have increased in efficiency and capacity, wall cranes, column cranes and monorail hoists are being used to an increasing extent. The development of monorail systems has been especially marked.

The improvement in the construction of the shops themselves cannot be overlooked. The increasing use of metal frames and sashes has resulted in much better lighting systems, and the ventilation facilities have also greatly improved. In this connection the efficient plant of the St. Lawrence Bridge Co., which was built and equipped expressly for the construction of the new Quebec Bridge, is worthy of note. The large number of special and efficient machines contained in this plant are of particular interest to anyone interested in the fabrication of steelwork. Considering the economic side only, it is hardly probable that the building of such an elaborate plant as this for the construction of a single bridge is justifiable.

Neglected Factors in the Designing of Masonry Dams.

Dam designers are, largely, men of contented mind. Failures of dams—one or two a year—occur and the dam builder applies his old formulas and marshals his old data and assumptions and designs anew. Since, ten or more years ago, the Austin, Texas, dam slid down stream and broke into pieces there have been prominent dam failures "due to foundation" but designers are still designing with scarcely more precise knowledge of rock friction and of uplift pressure than they had two decades ago. They have discussed these factors and that of ice thrust voluminously but they have accumulated very few data.

A contributor in this issue calls for action toward determining some of the neglected factors in the designing of masonry dams. Because his remarks are brief and because they are a justified demand for facts we urge engineers to read them. Without facts, we are inclined to agree with a recent writer that there has been enough discussion of dam design. But there is a real need for the accumulation and discussion of facts. Discussion affirming and denying the importance of uplift pressures would if gathered make a fair sized book, but there is not a test record showing what the intensity and distribution of such pressures may be or indeed showing beyond question that any uplift pressures exist in cases of rock founded dams. Accounts are plentiful of rock sliding on rock under dams but there are no test records that are not old and unreliable of the coefficient of friction between rock and rock.

Two years ago (E. & C., June 26, 1912) we published a summary of the practice and opinions of many dam designers in respect to ice thrust on dams. In the same article we published the results of the most complete field and laboratory study of ice pressures and ice strengths that, we believe, has ever been prosecuted. Two years have added no fact of importance to these records and summaries yet they made plain that most encouraging results would probably follow from continued investigation.

Investigation is needed of the character and intensity of ice pressure and of uplift pressure and it is needed to determine the friction of rock on rock. It is needed whether or not it turns out that engineers may safely neglect specific allowance for these factors in designing dams. Their significance or insignificance will never be agreed upon until one or the other is demonstrated. Ice pressure, uplift and rock friction are all capable of determination to some degree of precision by means of not difficult laboratory and field tests. This journal has repeatedly urged that such tests be undertaken and it here urges again that they be undertaken.

Buildings

Some Data on the Use and Properties of Cinder Concrete Floors.

The following article on cinder concrete floors considers: the manner of using reinforced cinder concrete floor slabs by various cities in the eastern part of the country; the unit stresses adopted by several cities, and some data tending to establish unit working stresses from actual stresses; the qualifications of this material for floors, together with a comparison with stone concrete; the arbitrary manner of testing floor constructions; and the difference between the conditions of such tests and the actual loadings found in buildings. The data were abstracted from a paper "Cinder Concrete Floors," by Guy B. Waite, in Proceedings, American Society of Civil Engineers, Vol. XL., p. 889.

Where cinders of good quality can be obtained cinder concrete, reinforced with steel, is used very extensively in the eastern part of this country for fireproof floor construction. It is estimated that more than 50 per cent of the large fireproof structures of Philadelphia have reinforced cinder concrete floor slabs, and New York probably has a larger proportion of such floors.

Floor slabs of stone and gravel concrete, reinforced with steel, of the same general form as cinder concrete have been systematized by engineers so that they are being used on a rational basis; but cinder concrete has been allowed to straggle along, without much attention from engineers in general, and is being used in a varied manner. Most large cities, such as Philadelphia, Boston and Chicago require cinder concrete for floor slabs to be designated, in thickness and in quantity of reinforcement, with certain values of unit stresses as a limit.

In New York and in most small cities in the eastern part of the country unit stresses are not considered in the use of such concrete. In these places its use is either entirely arbitrary or a separate test is required for each condition.

In New York, since the beginning of the use of cinder concrete floors in 1896, there have been about 200 approvals for reinforced floor slabs of this material, based on load tests. These approvals were for varying spans, thicknesses of concrete, and quantities of reinforcement. The tests have generally been conducted with a uniformly distributed load (of pig iron, bags of cement, or sand) imposed on the slab.

As will be shown the results of former tests, which include all principal constructions in use, are extremely variable. The writer has been informed that since 1911 concentrated load tests are required for all new approvals by the Borough of Manhattan. The result is that there are approvals of concrete floor slabs in which the effective depth is 3 ins. (4 ins. total thickness) and the reinforcement is less than ½-lb. per square foot on an 8-ft. span, with a live load of 250 lbs. per square foot. In these slabs the concrete is in the proportion of 1:2:5.

Each contractor in New York is held strictly to the details of construction for his approvals. For instance, if the approval allows a live load of 250 lbs. per square foot up to 6-ft. spans, and it is desired to use the construction for a live load of only 60 lbs. per square foot on 6-ft. 6-in. spans, this construction will not be allowed although stressed less than with the approved loads.

Moreover, stone and gravel reinforced concrete floor slabs are required to be designed according to the following values: Extreme fiber stress in concrete, 650 lbs. per square inch; maximum tension in mild steel, 16,000 lbs. per square inch; ratio of moduli, 15. According to these values, stone concrete slabs in some cases would be designed with a greater thickness and have greater reinforcement than the approvals for cinder concrete.

The writer believes that the excuse "that cinder concrete is so extremely variable as not to permit of the assignment of unit stresses" is not well founded. The fact that cinder concrete is being used extensively in engineering construction is sufficient reason for assigning, for safety, some unit stresses, in order to insure uniformity. The present manner of making arbitrary approvals has led to confusion, various forms of reinforcement obtaining approvals in which the spans, thicknesses of concrete, quantities of reinforcement and floor loads vary beyond description.

The proposed building codes of 1912 and 1913 have persistently continued the present manner of using cinder concrete (with slight modifications) for floor constructions. In these codes the manner of designing and constructing cinder concrete floor slabs is arbitrarily specified. In the proposed code of 1913 the allowable carrying capacity of the slab, with a 3-in. effective depth on 8-ft. spans, is increased (over the 1912 code) to a live load of 250 lbs. per square foot.

CINDERS.

Cinders for concrete floor construction are understood to mean either hard or soft coal ashes coming from boiler plants. Coal ashes from a stove or a small house boiler are generally too finely burned to be of use in this kind of concrete. Cinders from either hard or soft coal having sharp particles greatly in excess of the smooth ashes make a concrete of considerable strength and of good fire-resisting qualities.

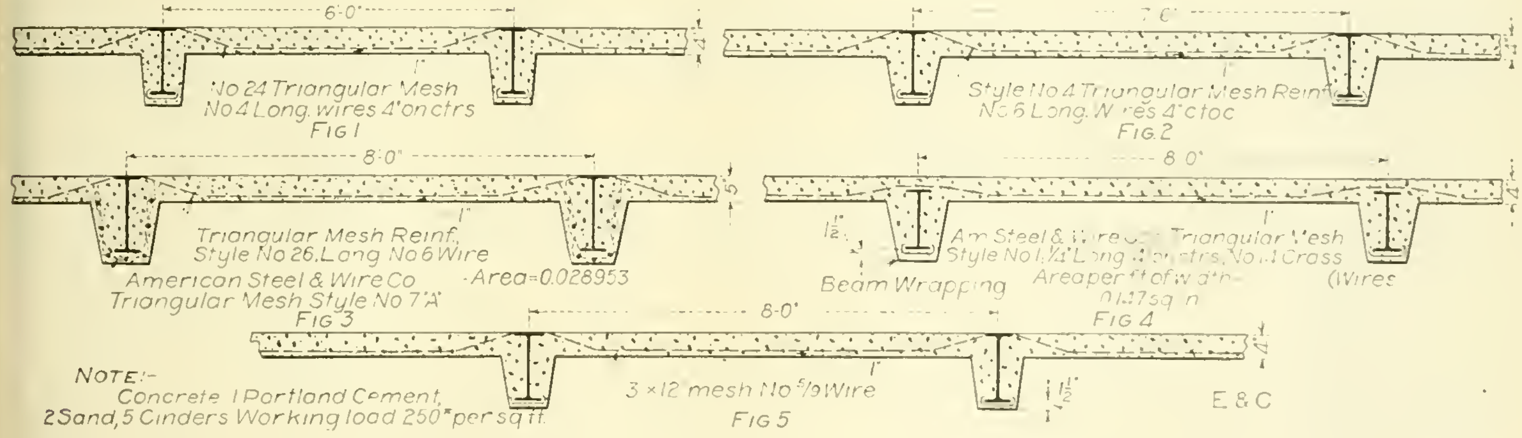
Cinders vary in size from the consistency of coarse building sand to clinkers several inches in diameter. Some hard-coal cinders run so perfectly in grade that good concrete is made without the addition of sand; more often, however, sand must be added. The quantity of sand required to fill the voids is variable, but the usual specification is 2 parts of sand to 5 parts of cinders.

After a long experience in the use of cinder concrete the writer believes that there should be no danger in assigning working stresses to the material, these stresses being taken from tests on known mixtures in which cinders of the lowest grade permitted are used. If the material is too poor to have such working stresses assigned it seems evident that cinder concrete is not a safe material for such an important part of the structure as the floor slab.

CINDER CONCRETE AS FIREPROOFING.

Since 1896 about 82 different floor constructions have been tested in the United States by fire and water. Nearly all these tests were made under the auspices of the Bureau of Buildings of New York, on full-sized floor slabs. About 40 of the tests were on reinforced cinder concrete floor slabs, about 23 on stone or gravel concrete floor slabs, 10 on some form of hollow tile, and the others on special floor constructions.

The test structures were about 14 ft. square and 9 ft. high, with a fire grate at the bottom, the floor slabs to be tested forming the ceiling. The average temperature on the tested floor slabs was 1,700° F., maintained for 4 hours. Following this there was a water test of 60 lbs. pressure through a 1⅛-in. nozzle applied for 10 minutes to the under side of the slabs. During the fire test a load of 150 lbs. per square foot remained on the entire surface of the slabs. After the fire test the slabs were subjected to a total load of 600 lbs. per square foot.


6'-0"
4"
1"
No 24 Triangular Mesh
No 4 Long. wires 4" on ctrs
Fig 1
7'-0"
4"
1"
Style No 4 Triangular Mesh Reinf.
No 6 Long. Wires 4" c to c
Fig. 2
8'-0"
5"
1"
Triangular Mesh Reinf.
Style No 26. Long No 6 Wire
American Steel & Wire Co
Triangular Mesh Style No 7 'A'
Area=0.028953
Fig 3
8'-0"
4"
1"
1½"
Am Steel & Wire Co Triangular Mesh
Style No 1, ¼" Long 4" ctrs, No 14 Cross
(Wires
Beam Wrapping
Area per ft of width
0.1475 sq in
Fig 4
8'-0"
4"
1"
1½"
3×12 mesh No 5/9 Wire
Fig 5
Note:-
Concrete 1 Portland Cement,
2 Sand, 5 Cinders Working load 250# per sq ft.
E & C

Figs. 1 to 5. Cinder Concrete Floor Slabs Approved by Bureau of Buildings, New York.

In these tests the cinder concrete floor constructions withstood the fire and water better than any other material. In the writer's opinion the reason for this is not due to the fact that cinder concrete is more fireproof than the others, but simply to the fact that it is not ruptured and destroyed by the expansions and contractions caused by fire and water.

Hollow tile, as a material, may be more fireproof than cinder concrete, but the stresses due to the expansion of the exposed surface destroy it.

TABLE I.—REINFORCEMENT FOR CINDER CONCRETE SLABS.
(From proposed Building Code of 1913, Sec 104, Art. 8.)
Tension wires 4 ins. on centers.

Live load per square foot of floor area.	Type of reinforcement.	Span between steel beams. 6 ft. 0 in. or less.		6 ft. 1 in. to 7 ft. 0 in.		7 ft. 1 in. to 8 ft. 0 in.	
			Area of steel per foot width of slab.		Area of steel per foot width of slab.		Area of steel per foot width of slab.
100 lbs. and less	Bars	0.4 lb.	(0.118 sq. in.)	0.5 lb.	(0.147 sq. in.)	0.6 lb.	(0.177 sq. in.)
	Wire mesh	No. 7 wire	(0.0735 sq. in.)	No. 6 wire	(0.0868 sq. in.)	No. 5 wire	(0.101 sq. in.)
101 to 150 lbs.	Bars	0.5 lb.	(0.147 sq. in.)	0.6 lb.	(0.177 sq. in.)	0.8 lb.	(0.236 sq. in.)
	Wire mesh	No. 6 wire	(0.0868 sq. in.)	No. 5 wire	(0.101 sq. in.)	No. 4 wire	(0.120 sq. in.)
151 to 200 lbs.	Bars	0.7 lb.	(0.206 sq. in.)	0.9 lb.	(0.265 sq. in.)	1.1 lb.	(0.324 sq. in.)
	Wire mesh	No. 5 wire	(0.101 sq. in.)	No. 4 wire	(0.120 sq. in.)	No. 3 wire	(0.140 sq. in.)
201 to 250 lbs.	Bars	0.8 lb.	(0.236 sq. in.)	1.0 lb.	(0.294 sq. in.)	1.2 lb.	(0.353 sq. in.)
	Wire mesh.	No. 4 wire	(0.120 sq. in.)	No. 3 wire	(0.140 sq. in.)	No. 2 wire	(0.162 sq. in.)

Matter in parentheses added for comparison. All other matter as per code.

One test structure which was built entirely of cinder concrete, mixed 1 to 4 without sand, withstood four different fires before being torn down to make room for improvements. Only the surface, for a depth of less than 1 in., was affected; this was dehydrated, but the burned cinder concrete was still intact and protected the remainder of the material. In this concrete there were at least 5 per cent of unburned pea coal, which was unaffected by the fire.

ARBITRARY TESTING OF CINDER CONCRETE.

In New York, after each type of cinder concrete construction had qualified for fireproofing, the variations in spans, carrying capacity, etc., were determined by each manufacturer by constructing the slabs he proposed to use and testing them with uniformly distributed loads under the supervision of the Bureau of Buildings. This was the method used for the approvals of the constructions shown in Figs. 1 to Fig. 5, inclusive, with the exception of Fig. 3, which was constructed for fire test. These constructions were taken from the files of the Bureau of Buildings from among some 200 similar approvals.

For comparison with the approved cinder concrete floor slabs shown by Figs. 1 to 5 the writer gives the relative thickness and the quantity of reinforcement for slabs under similar conditions according to the unit stresses used by Philadelphia (see Figs. 6 to 8), and also the thicknesses and quantity of reinforcement required by New York for 1:2:4 stone concrete (see Figs. 9 to 11).

TABLE II.—UNIT STRESSES FOR CINDER CONCRETE AS USED BY VARIOUS CITY DEPARTMENTS.

City.	Extreme fiber stress, in lbs. per sq. in.	Ratio of moduli of elasticity.	Kind of cinders used.	Mixture.	Work'g stresses in steel, in lbs. per sq. in.	Preferred reinforcement.
Philadelphia	250	30	Hard coal.	1:2:4	16,000	None.
Boston	300	25	Soft coal.	1:2:4 to 1:2½:5	16,000	None.
Chicago	245	30	Hard or soft.	1 to 5 aggregate.	18,000	None.
Baltimore	300	30	Hard or soft.	1:2:4	15,000	None.

It is to be noted in comparing the stone or gravel concrete construction (Figs. 9 to 11) with the cinder concrete that the former is made unquestionably continuous, and the latter, in most cases, is cut into and is simply supported by the steel beams (Figs. 1 to 8 and 12 and 13).

The approvals for the longest spans with the highest carrying capacity were based on tests with slabs which were continuous over supports (Figs. 1 to 5). In practice it is seldom that cinder concrete floor slabs between steel beams can be made continuous throughout a floor.

In Figs. 12 and 13, which are assumptions for illustration, it will be noticed that the reinforcement is simply laid over the top flanges of the supporting steel beams. In practice there is no way of taking up the slack in the reinforcement coming over the tops of the beams, and the consequences are that under such conditions there can be no continuous action—such as engineers understand—in the combined steel and concrete structure. Unless considerable slack is allowed on the sides of each supporting beam the tamping of the concrete works the reinforcement to the top of the slab.

In buildings where a wooden floor is used the sleepers are generally laid directly on the steel beams, and consequently the concrete is kept below the top flanges of the beams. Here, therefore, the concrete is not continuous in any part of the floor; but in the interior spaces on all floors, whether continuous or not, the channels on the outside spans and against openings in floors cannot have either concrete or reinforcement continuous over them (Figs. 12 and 13).

To any one acquainted with the actual construction of cinder concrete floors it must be apparent that some parts of each floor will not admit of being constructed with the same care that might be attained in a test construction; therefore it does not follow that because a slab was continuous in the test it is so in actual practice.

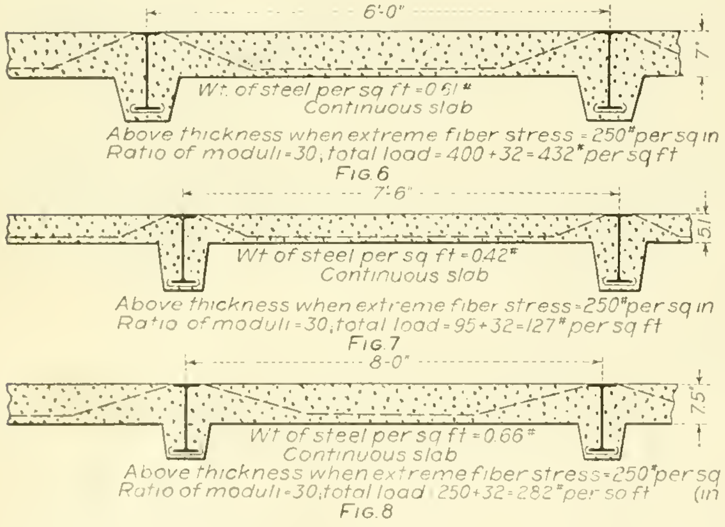
Moreover, these tested slabs were held by adjacent slabs on each side of the supports, thus enabling each side to take thrust. In practice the outside slabs next to walls, elevator shafts, stairways, show windows, etc., are only resisted for thrust by tie rods and by concrete slabs on one side. If the writer is correct in his statement that outside slabs, etc., are not under the same conditions as the tested ones, where the reinforcement and the concrete is carefully made continuous, then this form of so-called continuous construction is being used in an unapproved and unsafe manner.

TABLE III.—WEIGHT AND COMPRESSIVE STRENGTH OF CINDER CONCRETE AS USED IN FIREPROOF FLOORS IN NEW YORK.

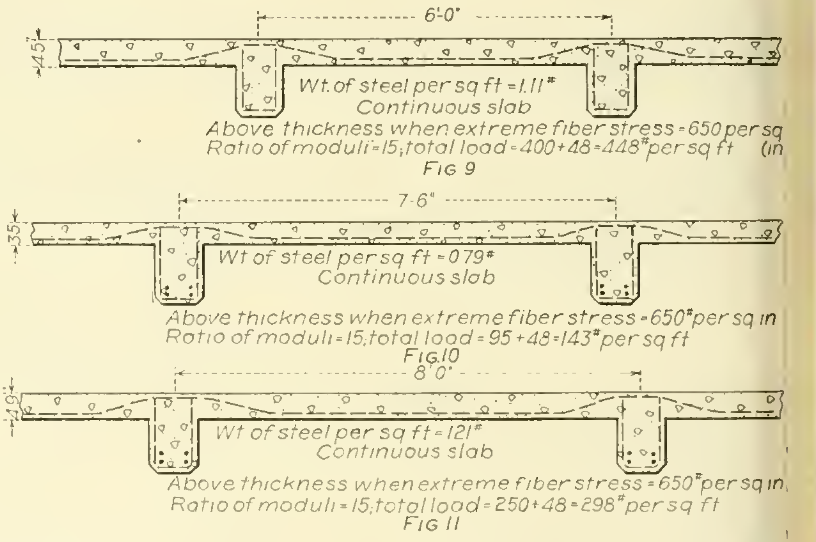
(The table covers 120 samples. Each figure given is the average of 10 samples.)

Description.	A.	B.	B2.	C.
Mix	1:2:5	1:1:5	1:2:5	1:2:5
Weight, lbs. per cu. ft.	107	100	107	109
One-month test:				
Crushing strength, lbs. per sq. in.	407	507	818	980
Mod. of elast., lbs. per sq. in.	924,600	857,400	1,230,000	1,492,000
Two-month test:				
Crushing strength, lbs. per sq. in.	701	662	1,254	1,035
Mod. of elast., lbs. per sq. in.	1,134,000	1,030,000	1,740,000	1,428,250
Six-month test:				
Crushing strength, lbs. per sq. in.	933	754	1,744	1,478
Mod. of elast., lbs. per sq. in.	971,000	1,050,000	1,348,000	1,276,000

Note.—B was hand-mixed; A, B2 and C were machine-mixed. The modulus of elasticity was determined at a point of the elastic curve corresponding to one-fourth the ultimate strength.


6'-0"
7"
Wt. of steel per sq ft = 0.61#
Continuous slab
Above thickness when extreme fiber stress = 250#per sq in
Ratio of moduli = 30; total load = 400+32=432#per sq ft
Fig.6
7'-6"
5.1"
Wt of steel per sq ft = 0.42#
Continuous slab
Above thickness when extreme fiber stress = 250#per sq in
Ratio of moduli = 30; total load = 95+32=127#per sq ft
Fig.7
8'-0"
7.5"
Wt of steel per sq ft = 0.66#
Continuous slab
Above thickness when extreme fiber stress = 250#per sq
Ratio of moduli = 30; total load 250+32=282#per sq ft (in
Fig.8

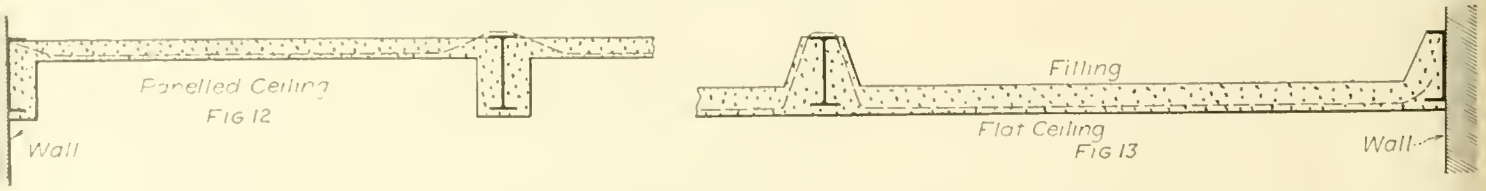
Figs. 6 to 8. Thicknesses and Reinforcements for Cinder Concrete Slabs Using Unit Stresses Specified by Philadelphia—Designed for Conditions Similar to Those for Slabs Shown in Figs. 1 to 5.


6'-0"
4.5"
Wt. of steel per sq ft = 1.11#
Continuous slab
Above thickness when extreme fiber stress = 650#per sq
Ratio of moduli = 15; total load = 400+48=448#per sq ft (in
Fig 9
7'-6"
3.5"
Wt of steel per sq ft = 0.79#
Continuous slab
Above thickness when extreme fiber stress = 650#per sq in
Ratio of moduli = 15; total load = 95+48=143#per sq ft
Fig.10
8'0"
4.9"
Wt of steel per sq ft = 1.21#
Continuous slab
Above thickness when extreme fiber stress = 650#per sq in
Ratio of moduli = 15; total load = 250+48=298#per sq ft
Fig 11

Figs. 9 to 11. Thicknesses and Reinforcements Required by Bureau of Buildings, New York, for 1:2:4 Stone Concrete—Designed for Conditions Similar to Those for Slabs Shown in Figs. 1 to 5.

With the conditions of continuous construction under which these slabs were tested, with a possible arching of the loading, and with supports braced against side flexure, the stress that came on the steel and concrete at the middle of the slab can only be guessed. Still, according to the latest proposed building code (that of 1913) tests and approvals of that kind were to have preference over unit stresses. The following is quoted from Sec. 104, Art. 8, of this proposed code:

> The concrete is made 1:2:5; slab thickness, 4 ins., with reinforcement 1 in. from bottom. Reinforcement must be as per tabulations.



Figs. 12 and 13. Assumed Designs to Which Reference is Made—Reinforcement Not Continuous.

It will be noticed from Table I that when the reinforcement is of wire mesh the area required is only about one-half as great as for any other form of reinforcement—even with the same kind of wire but without the mesh.

USE OF CINDER CONCRETE BY CITIES OTHER THAN NEW YORK.

The cities mentioned in Table II use a ratio of moduli of elasticity of about 30, and the highest allowed extreme fiber stress in any of these cities is 300 lbs. per square inch.

The results of tests on cinder concrete given in Table III are those obtained from some tests conducted at Columbia University with the co-operation of the Bureau of Buildings of New York. Although these limited tests would seem to show that a ratio of moduli of less than 30 could be used the writer has been informed that specimens "C" were from extra good concrete, and that "A" and "B" are nearer the average material. The ratios used by Philadelphia, Chicago and Baltimore seem to be about right. As will be seen later a less ratio of moduli will give a less carrying capacity, other things remaining the same.

Based on the tests made at Columbia University it is the feeling, at the present stage of these tests, that an extreme fiber stress of from 200 to 250 lbs. per square inch on cinder concrete is all that can be used conservatively. Only the tests up to six months old have been made; the final tests may alter this assumption, but for discussion these values are sufficiently correct.

DESIGNING REINFORCED CONCRETE FLOOR SLABS.

Solid slabs of cinder concrete are not weak in shear, so that the discussion of vertical and diagonal shear will not be necessary. In the writer's observation of many breaking tests on cinder concrete floor slabs there has never been a failure near the supports; almost uniformly, the breaking has taken place near the center of the slab. Usually the top of the central portion has spalled off (cup fashion), apparently due to horizontal shear and compression, after which the slab suddenly collapsed.

Adhesion.—The adhesion of cinder concrete to various kinds of reinforcement is shown by Table IV, which gives the results of a series of tests on 1:2:4 cinder concrete, made in 1904 by H. B. Gaylord and H. A. Pratt, at Stevens Institute of Technology. The cement was Lehigh Portland, and the cinders were those coming from a boiler plant. The concrete blocks were 6 x 6 ins. and the steel was embedded in the center.

It will be noted by referring to Table IV that the plain bars held in tension quite as well as the deformed bars, the average adhesion of the surface embedded being about 200 lbs. per square inch. This same fact, as to the relative adhesion of plain and deformed steel bars in stone concrete, has already been shown in numerous tests. It is therefore considered that the form of the reinforcement is immaterial so long as it has sufficient surface embedded.

(The author gives an analysis of cinder reinforced concrete beams, and determines the thickness of slabs and the quantity of steel required for various assumptions, which we have omitted in our abstract.)

STRESSES IN APPROVED FLOOR SLABS.

The location of the neutral axis for any particular thickness of slab is a function of the relative stresses in the materials and of the moduli of elasticity. Referring to Fig. 5, assume that the ratio of moduli remains constant at 30, and that $\frac{f_s}{f_c} = 64$, i. e., $\frac{16,000}{250}$ or $\frac{20,000}{312.5}$, and that the slab tested is continuous

The neutral axis is 0.32*d* from the top of the slab, when the concrete and steel are proportioned for these stresses. For a 4-in. slab, therefore, the neutral axis would be 0.96 in. from the top of the slab. A normal 4-in. slab is good for a total floor load of 150 lbs. per square foot on a 5-ft. span with a reinforcement of 0.31 lb. per square foot; but in this 4-in. slab of 8-ft. span, as approved, No. 5 wires at 3 ins. on centers are used. This makes the actual reinforcement about 50 per cent greater than the normal reinforcement based on the above values.

TABLE IV.—RESULTS OF ADHESION TESTS ON 1:2:4 CINDER CONCRETE.

No. of specimen.	Length embedded, in inches.	Perimeter, in inches.	Area of cross-section, in square inches.	Shape of bar.	Kind of concrete.	Pull, in pounds per inch.	Pull, per inch of perimeter.	Pull per square inch.
1	6¼	1.75	0.094	De Mann	Cinder, 1:2:4	396	1,414	206.2
2	11⅜	1.75	0.094	"	"	598	3,885	247.2
3	6¼	1.75	0.094	Flat.	"	376	1,343	214.6
4	16	1.75	0.094	"	"	250	2,285	142.8
5	18	1.75	0.094	"	"	230	2,371	131.4
6	23	1.75	0.094	"	"	176	2,072	89.8
7	5	1.57	0.196	Round.	"	400	2,730	255.0
8	6⅛	1.57	0.196	"	"	553	2,076	339.2
9	9	1.57	0.196	"	"	647	3,710	412.2
10	13	1.57	0.196	"	"	379	5,460	196.0
11	16	1.57	0.196	"	"	417	4,331	277.0
12	28¾	1.57	0.196	"	"	239	4,188	145.0
13	6	1.625	0.156	½ by 5/16-in.	"	500	1,834	307.6
14	10¼	1.625	0.156	"	"	146	914	90.0
15	12	1.625	0.156	"	"	500	3,668	308.9
16	17¼	1.625	0.156	"	"	345	3,661	212.3
17	19¾	1.625	0.156	"	"	310	3,778	191.0
18	27¼	1.625	0.156	"	"	232	3,895	143.0
19	4¾	3.25	0.625	1 by ⅝-in.	"	990	1,477	311.0
20	6¾	3.25	0.625	"	"	800	1,661	246.3
21	14¾	3.25	0.625	"	"	665	3,015	204.6
22	20¼	3.25	0.625	"	"	680	4,353	215.0
23	26¾	3.25	0.625	"	"	645	5,303	193.0
24	11½	3.125	0.203	1 by ⅜-in. channel.	Stone.	895	3,168	247.9
25	18	3.125	0.203	"	"	610	3,505	194.7
26	22½	3.125	0.203	"	"	523	3,433	103.8
27	6⅜	3.125	0.203	"	Cinder.	840	1,715	273.1
28	8¼	3.125	0.203	"	"	848	2,304	271.1
29	16¾	3.125	0.203	"	"	245	1,120	166.8
30	17	3.125	0.203	"	"	555	3,019	177.6
31	20¼	3.125	0.203	"	"	435	2,216	141.0
32	22½	3.125	0.203	"	"	476	3,435	154.0
33	35¾	3.125	0.203	"	"	291	3,357	93.9
34	6½	1.00	0.0625	Ransome.	"	431	2,800	439.8
35	7½	1.00	0.0625	"	"	377	2,825	376.6
36	10	1.00	0.0625	"	"	409	4,085	408.5
37	13¼	1.00	0.0625	"	"	334	4,435	334.8
38	22	1.00	0.0625	"	"	270	5,250	238.6
39	26½	1.00	0.0625	"	"	755	2,000	75.4
40	6	1.00	0.0625	Square.	"	412	2,650	441.6
41	7¾	1.00	0.0625	"	"	322	2,500	332.5
42	11½	1.00	0.0625	"	"	322	2,500	217.4
43	15½	1.00	0.0625	"	"	261	3,200	206.4
44	25¼	1.00	0.0625	"	"	128	3,215	127.3
45	32¾	1.00	0.0625	"	"	113	3,650	111.1

This relative excess of steel (reducing the comparative stress coming on the reinforcement) will lower the neutral axis below that given above. The stress in the steel, $\frac{f_s}{E_s}$, for this case is $\frac{2}{3} \times \frac{16,000}{30,000,000}$. On account of the excess steel the neutral axis will be located 0.41*d* or 1.23 in. from the top of the slab—instead of 0.96 in. for the case where there is no excess of steel.

According to the ruling of the New York Building Department the 4 ins. of cinder concrete weighs 32 lbs. per square foot. Therefore the approval of this floor construction is for a total load of 250 + 32 = 282 lbs. per square foot.

If we considered the slab continuous the bending moment is $\frac{1}{12} WL^2 = 18,048$ in. lbs.

This is equal to the resisting moment (one force of the couple being $12 f_c \frac{1.23}{2}$) having a lever arm equal to $\frac{2}{3} \times 1.23 + 1.77 =$ 19.11 × f_c in.-lbs. Therefore f_c = 944 lbs. per square inch, which is the extreme fiber stress on the concrete. The area of the reinforcement is 0.134 sq. in., and f_s = 52,000 lbs. per square inch.

If instead of considering the slab continuous we take an end slab, as is done with stone concrete, we have a moment of $\frac{12 WL^2}{10}$, or 21.658 in.-lbs. In this case the extreme fiber stress will be 1,133 lbs. per square inch, and the stress on the steel, 62,000 lbs. per square inch.

If we take the condition of reinforcement simply laid over the supports (as shown in Figs. 12 and 13) then the slab is simply supported at the ends, and the moment is $\frac{12 WL^2}{8}$, or 27,072 in.-lbs. This latter condition gives an extreme fiber stress of 1,417 lbs. per square inch, and a steel stress of 78,000 lbs. per square inch.

Under such abnormal stresses the modulus of elasticity for cinder concrete probably does not remain constant, which would slightly modify the foregoing deductions. However, even if an indefinite number of assumptions were made regarding the ratio of moduli of elasticities of wire and of cinder concrete, ratios of stresses in concrete and steel, and the variable support of slabs, etc., the foregoing deduced stresses would still be found to represent approximately correct conditions.

The writer wishes to call attention to the comparison of the thickness of stone concrete slabs with approved cinder concrete slabs under similar conditions. It is found that, for the same conditions as for the approved cinder concrete slab (Fig. 5) discussed herein, the stone concrete would be required to be about 1 in. thicker than the cinder concrete.

RECOMMENDATIONS

(1) The practice of testing cinder concrete, arbitrarily, under the most favorable conditions, and then recommending the results of these tests to be used under the most unfavorable conditions which may exist in a building, is dangerous and entirely unnecessary.

(2) Unit stresses for the design of reinforced cinder concrete are of more importance for its proper use than the unit stresses used for reinforced stone concrete, because of the greater variableness of the former.

(3) There are at present sufficient precedents for choosing some unit stresses for the use of reinforced cinder concrete slabs in each locality where this material is being used.

(1) It is practical and of little cost for each city to establish ultimately proper working stresses for reinforced cinder concrete, based on tests with the materials used in its locality.

Results of Tests to Determine the Mortar-Making Qualities and Characteristics of Illinois Sands.

The importance of securing a good quality of sand for use in mortar and concrete is not always appreciated, in fact, the sand is often selected after only a superficial examination. As a result sands which produce a mortar deficient in both durability and strength are often used when sands of a high quality are easily obtainable. There is need of more reliable data on the characteristics and mortar-making qualities of typical sands. The following data on a number of sands found in various parts of Illinois are of value in pointing out their characteristics, the data being abstracted from Bulletin No. 70, Illinois Engineering Experiment Station, by C. C. Wiley. The results of the tests and investigations here recorded are directly applicable to mortar for stone or brick masonry, and in all probably apply equally well to concrete. Many of the conclusions drawn from the tests are applicable to sands in general.

Many of the samples of sand were furnished by city engineers throughout Illinois. The following list gives the cities whose engineers furnished samples of sands for these tests, together with the reference numbers used to designate the various samples:

City.	Ref. No. of sand.	City.	Ref. No. of sand.
Aurora	17	Galesburg	13
Beardstown	27	Jacksonville	26
Bloomington	7	Joliet	18
Cairo	25	Moline	19
Champaign	12	Mt. Carmel	28
Chicago	1, 2, 3, 4	Paris	23
Decatur	14	Rockford	8
East St. Louis	21	Springfield	9, 10, 11
Elgin	5, 6	Taylorville	24
Freeport	15, 16	Waukegan	22

DESCRIPTION OF SANDS.

The following description of the sands used in these tests is based partly on the general information furnished by the parties who sent the sands and partly on the tests. Samples Nos. 20, 29, 30, 31 and 32 were collected by those in charge of the tests:

No. 0.—This is the standard sand used in the comparative tests of cement. It is pure quartz, and comes from the St. Peter formation near Ottawa, Ill. The grains are spherical, almost transparent, with dull surfaces, and are nearly of the same size, since the sand is screened to pass a No. 20 sieve and be retained on a No. 30 sieve.

No. 1.—This sand was taken from banks along the shore of Lake Michigan. Its color is light yellow, and the grains are angular and consist principally of quartz. The sand is very fine, and contains 0.2 per cent of suspended matter.

No. 2.—This is also a bank sand from the shores of Lake Michigan. It is light gray in color and is composed principally of quartz with a small proportion of granite, flint and limestone. It is somewhat better graded than No. 1 and contains 0.3 per cent of suspended matter.

No. 3.—This is another lake sand. The smaller grains are principally quartz, while the larger ones are composed of granite, flint, limestone and chert, which gives the sand its light gray color. It is better graded than either No. 1 or No. 2. It contains 0.3 per cent of suspended matter.

No. 4.—This is a sample of Joliet limestone screenings. The stone is very close and even grained and has a pale blue tint. In crushing, some of it passes into a very fine dust, but it contains no suspended matter. The sample is hardly typical.

No. 5.—A bank sand from the Hammond pit, near Elgin, light yellowish gray in color, contains less quartz than the average Illinois sand; also contains limestone, flint and chert, one of the coarsest sands tested, although reported as washed sand it contains 1 per cent of suspended matter.

No. 6.—Screened sand from Stimpson pit near Elgin, grayish in color, and contains a small proportion of flint, granite and chert, although consisting mostly of quartz, somewhat finer than No. 5, contains 0.5 per cent of suspended matter.

No. 7.—Bank sand from Sugar Creek, near Bloomington, quite uniform in quality; is much used for pavement cushions, but is considered too dirty for concrete; contains 8 per cent of suspended matter in the form of clay which is distributed through the sand and does not form a coat on the grains.

No. 8.—Bank sand from near Rockford; deposits are large and of uniform quality; sand is light gray in color, and contains considerable limestone and flint; sand is well graded; contains 0.6 per cent of suspended matter.

No. 9.—Sample of sand used in construction of Supreme Court building at Springfield; it is a bank sand from near Lincoln; has dark gray color, as about one-half of its mass consists of dark colored flints and granites, the remainder being quartz with some limestone; is fairly well graded and contains 1.1 per cent of clay which adheres to the grains.

No. 10.—Bar sand from Mississippi River, near Alton; dark gray in color, and contains limestone and flint; has 0.3 per cent of suspended matter; sand is very fine.

No. 11.—Bar sand from near Alton; brownish gray in color, and contains some flint and limestone; finer grains are well rounded, but coarser ones are quite angular; very fine but not quite as fine as No. 10; contains 0.2 per cent of suspended matter.

No. 12.—Bank sand from along Wabash River, near Covington, Ind.; yellowish gray in color, and contains some limestone, flint and shale; grains all well rounded; fairly well graded; contains 1.5 per cent of suspended matter in form of loose clay.

No. 13.—Bank sand from near Gladstone; has gray color, and contains some flint and granite, with grains moderately angular; is quite fine, and is the cleanest of the bank sands, containing only 0.2 per cent of suspended matter.

No. 14.—Screened sand from bank gravel found along Sangamon River, near Decatur; has brownish gray color, and contains considerable flint, granite, limestone and shale, with some softer rocks containing iron oxide; is the coarsest sample of natural sand tested; contains 2.5 per cent of suspended matter.

No. 15.—Bank sand from near Freeport; is yellow in color due to 1.3 per cent of clay which adheres tightly to grains; is composed principally of quartz, with some flint, limestone and granite; only moderately fine; grains are quite hard and well rounded.

No. 16.—Sandstone screenings from near Freeport; stone is rather soft; sample is deficient in fine material; contains no suspended matter.

No. 17.—Screened sand from bank gravel near Aurora; has yellowish color, and is composed almost entirely of quartz, with some shale; grains are well rounded, and the 0.5 per cent of suspended matter adheres to them but is easily removed by wetting; comparatively coarse and not well graded.

No. 18.—This sand, which comes from the Fuller pit, near Joliet, has some rather unusual characteristics; color is reddish yellow, due to large amount of clay; grains are quite uniform and rounded; contains 18.3 per cent by weight of suspeoded matter, consisting almost entirely of clay which adheres tightly to grains and cannot be washed off without scrubbing; after a thorough washing the sand is quite different in appearance, being grayish in color; sand is about half quartz, the other materials being limestone and granite, with considerable shale and other soft rocks.

No. 19.—Bar sand from Mississippi River, a few miles above Moline; has grayish color, and is composed principally of quartz, with some flint, limestone and granite; is quite fine, contains no suspended matter, and has a decided deficiency of very fine material.

No. 20.—Bank sand from near Urbana; has yellowish gray color, and contains some granite, flint, shale and chert; grains are well rounded, and are coated to a considerable extent with clay, of which there are 3.5 per cent; grains are not well graded, when wet this sand behaves much like quicksand.

No. 21.—Sand was dredged from bed of Mississippi River at East St. Louis; has light gray color, and contains some granite and flint, but consists mostly of pure quartz, almost transparent; is the sharpest of the natural sands tested; not well graded but contains a very low percentage of voids; contains only a trace of suspended matter.

No. 22.—Sand from shore of Lake Michigan, near Waukegan; sample has a dark gray color, and consists principally of quartz, flint, limestone, granite, sandstone and shale, with a small amount of coal and cinders; sand is very fine, and contains no suspended matter.

No. 23.—Bank sand from uniform deposits near Paris, Ill.; sand is very fine and grains are well rounded, consisting principally of quartz, with some limestone and chert; color is pale yellow, sand contains 1.2 per cent of suspended matter.

No. 24.—Bank sand from uniform deposits near Taylorville; sand is light yellow in color, and the grains are well rounded; consists principally of quartz, limestone and shale; sand is the finest and lightest of any tested, and contains 1 per cent of suspended matter.

No. 25.—Sand pumped from bed of Ohio River, above Cairo, Ill.; has grayish color, and contains some granite and limestone; is moderately fine, but is deficient in both coarse and fine grains; contains 0.3 per cent of suspended matter.

No. 26.—Sand from bed of Mississippi River, near Hannibal, Mo.; has bluish color, and contains considerable flint, granite and chert, the grains being moderately rounded; deficient in both coarse and very fine grains, and contains 0.2 per cent of suspended matter.

No. 27.—Bank sand from somewhat variable deposits near Beardstown; color is reddish yellow, due to 4.4 per cent of clay which forms a coating around the grains; sand contains some granite, limestone, flint and considerable soft

TABLE I.—RESULTS OF TESTS TO DETERMINE THE TENSILE STRENGTH OF 1:3 MORTARS USING VARIOUS SANDS.

No.	Sand. Name.	Age, days.	Tensile strength—Pounds per sq. in. Individual briquettes.						Average.
00	Neat cement	7	615	645	640	690	620	...	642
		28	780	730	770	760	720	...	77
		90	755	820	785	780	...	...	78
0	Ottawa standard, 20-30	7	220	200	220	230	230	...	225
		28	310	300	315	340	315	295	313
		90	375	365	375	370	360	365	368
1	Chicago K. I. Co	7	115	110	108	113	108	...	111
		28	205	195	190	200	165	195	192
		90	210	210	210	190	205	...	205
2	Chicago Z. R. Co	7	150	175	175	145	150	...	159
		28	300	260	275	290	250	270	274
		90	315	310	330	290	...	...	311
3	Chicago K. I. Co	7	200	220	205	170	180	...	195
		28	265	295	260	270	305	290	281
		90	320	310	320	305	315	...	314
4	Joliet limestone screenings	7	220	240	200	210	220	...	218
		28	295	335	365	350	330	...	335
		90	450	530	505	460	...	...	485
5	Elgin, Hammond pit	7	245	270	240	220	250	...	245
		28	380	375	395	425	410	380	394
		90	465	455	450	445	490	480	464
6	Elgin, Stimpson pit	7	190	180	210	225	210	200	203
		28	340	340	360	350	390	...	356
		90	415	420	405	400	...	...	410
7	Bloomington	7	225	230	225	235	210	225	225
		28	225	265	250	240	235	260	246
		90	295	305	275	285	270	...	286
8	Rockford	7	250	240	260	230	245	265	248
		28	335	345	350	335	330	340	339
		90	385	380	410	360	395	385	386
9	Lincoln	7	235	220	230	210	200	205	217
		28	285	270	275	285	275	290	279
		90	310	310	290	300	300	305	303
10	Alton	7	185	180	175	185	150	175	175
		28	245	230	250	255	220	260	243
		90	275	270	260	270	270	270	269
11	Alton	7	180	195	210	195	190	...	194
		28	245	235	250	245	240	...	243
		90	270	285	295	280	270	280	280
12	Covington, Ind	7	205	185	200	200	205	195	198
		28	265	280	250	280	285	245	267
		90	315	315	310	305	315	300	310
13	Gladstone	7	190	180	180	170	170	180	178
		28	230	220	205	210	225	220	218
		90	250	235	240	230	240	250	241
14	Decatur	7	270	310	285	285	305	265	287
		28	380	375	355	355	360	...	365
		90	425	435	420	430	430	...	428
15	Freeport	7	245	205	200	220	190	225	214
		28	275	285	270	260	265	265	270
		90	335	330	320	315	310	295	317
16	Freeport sandstone screenings	7	180	180	195	220	170	195	190
		28	220	250	240	210	250	235	234
		90	235	260	270	270	300	280	269
17	Aurora	7	270	290	260	270	270	...	272
		28	390	415	400	435	410	...	410
		90	470	460	450	450	440	...	454
18	Joliet	7	120	130	98	100	100	100	108
		28	100	105	125	115	140	160	124
		90	145	170	155	150	150	175	159
18	Joliet (washed)	..	...	...	...	...	...	...	...
		28	150	170	190	175	185	185	176
		..	...	...	...	...	...	...	...
19	Moline	7	160	180	175	170	160	180	171
		28	250	225	235	245	...	...	239
		90	275	260	260	245	240	265	257
20	Urbana	7	190	185	190	200	200	195	193
		28	240	240	230	220	245	265	240
		90	365	320	315	310	340	330	330
21	East St. Louis	7	170	185	170	160	170	175	172
		28	195	210	205	190	210	230	207
		90	285	275	230	245	245	265	258
22	Waukegan	7	140	145	160	175	155	140	158
		28	235	230	225	225	230	220	228
		90	300	270	280	310	305	310	296
23	Paris	7	125	130	120	130	120	115	123
		28	200	200	200	205	190	190	198
		90	260	245	285	265	245	255	259
24	Taylorville	7	110	100	100	115	130	135	113
		28	140	155	150	155	135	160	149
		90	210	220	195	220	200	220	211
25	Cairo	7	155	155	140	155	175	140	153
		28	260	210	235	220	245	240	234
		90	300	295	300	325	280	285	298
26	Hannibal	7	160	150	150	175	175	180	165
		28	220	200	205	225	180	220	208
		90	290	270	290	275	275	275	279
27	Beardstown	7	95	80	90	90	90	70	86
		28	115	105	100	105	115	100	107
		90	145	130	150	135	135	145	140
28	Mt. Carmel	7	100	100	85	100	100	105	99
		28	140	135	130	140	125	140	135
		90	185	175	175	165	160	160	170
29	La Salle	7	130	120	130	110	100	110	117
		28	200	180	175	190	200	170	186
		90	175	200	185	190	210	200	194
30	Peoria	7	195	170	180	185	170	170	178
		28	200	205	200	205	210	210	205
		90	235	260	265	260	280	270	262
31	Peoria	7	165	140	175	185	150	185	167
		28	210	220	230	200	235	235	221
		90	315	320	295	285	320	300	306
32	Peoria	7	120	110	105	105	115	125	114
		28	155	155	145	145	160	160	153
		90	185	170	190	180	165	180	178

material, and shows a tendency to form lumps; next in fineness to No. 24.

No. 28.—Bar sand from Wabash River, near Mt. Carmel; color is yellowish, and sample contains flint, granite and cinders; almost as fine as No. 27; it is the finest and dirtiest of the river sands, containing 2.4 per cent of suspended matter; grains are well rounded.

No. 29. Bank sand from along Little Vermillion River, above La Salle; color is gray, and the sand consists almost entirely of quartz, the grains being well rounded; sand is very fine, and contains 3.2 per cent of suspended matter.

No. 30.—Sand from Cresent gravel pit near Peoria; color is yellowish and sand contains some shale, flint and granite; grains are well rounded and quite fine; sand contains 2.7 per cent of suspended matter.

No. 31.—Sand was taken from large sewer trench in Peoria and was used in the construction of the sewer; color is yellowish gray; sand contains some flint, granite, limestone and chert; it is quite fine and contains 1.9 per cent of suspended matter.

No. 32.—Bar sand from Illinois River, above Peoria; color is brownish gray; about one-half of the grains are quartz, the remainder being flint, limestone, granite, chert and cinders; sand is very fine, and contains 0.6 per cent of suspended matter.

DESCRIPTION OF TESTS.

Preparation of Samples.—The samples of sand were shipped to the laboratory in bags or in tight, paper-lined boxes. As soon as a sample was received it was given a "sample number" for identification. It was then screened to pass a wire sieve having square openings of 0.20 in., thoroughly dried over a steam coil, and then stored in wooden lockers in a room maintained at about 70° F. It will be noted that a maximum size of grain of 0.20 in. was used throughout these tests. This size was adopted at the beginning of the series of tests which was before the national engineering societies generally adopted 0.25 in. as the maximum size; and in the later tests it was thought unwise to make a change as it was impracticable to repeat some of the earlier tests. Fortunately the difference arising from using 0.20 instead of 0.25 is very small, since none of the sands contained an appreciable amount of material larger than 0.20 in.

TABLE II.—RESULTS OF SIEVE ANALYSIS TESTS OF SANDS.

Sand. No.	Name.	Susp. matter. per cent.	Per cent by weight passing sieve No.:— 200	150	100	74	60	40	30	20	16	10	8	5	0.2 in.
1	Chicago, K. I. Co	0.3	...	...	4.2	48.6	69.5	96.4	99.1	99.7	99.8	...	100.0	....	100.0
2	Chicago, Z.-R. Co	0.3	...	...	2.5	20.0	25.9	77.3	87.5	93.7	94.7	...	99.1	....	100.0
3	Chicago, K. I. Co	0.3	...	...	2.4	9.9	15.9	49.1	69.0	78.3	79.8	...	89.1	....	100.0
4	Limestone Scr	0.0	...	...	8.1	9.3	10.7	14.9	15.5	19.9	21.2	...	52.3	....	100.0
5	Elgin	1.0	...	...	2.8	5.0	7.4	20.9	36.4	51.7	55.0	...	84.0	....	100.0
6	Elgin	0.9	...	...	2.1	5.2	9.0	23.6	39.2	60.1	65.5	...	97.3	....	100.0
7	Bloomington	8.0	9.0	9.7	11.4	19.8	21.7	43.0	56.5	67.2	69.9	85.8	92.1	96.6	100.6
8	Rockford	0.6	...	...	1.7	5.5	10.8	25.1	34.3	60.0	68.2	...	98.0	....	100.0
9	Lincoln	1.1	...	...	2.4	4.1	6.5	26.7	46.4	61.7	65.8	..	95.4	....	100.0
10	Alton	0.3	...	...	1.2	5.2	10.5	45.6	70.2	87.8	90.3	...	98.2	....	100.0
11	Alton	0.2	...	...	2.2	5.3	8.9	30.1	53.8	76.2	80.0	...	97.0	....	100.0
12	Covington, Ind	1.5	...	...	3.6	5.7	7.6	19.8	31.6	47.9	53.3	...	93.5	....	100.0
13	Gladstone	0.2	...	...	1.1	6.8	12.9	63.8	88.8	97.0	97.9	...	99.8	....	100.0
14	Decatur	2.5	2.9	3.1	3.9	6.0	7.8	21.0	35.8	45.9	48.7	71.6	81.5	93.7	100.0
15	Freeport	1.3	3.0	3.7	6.6	14.4	18.5	43.9	63.6	79.2	82.5	97.2	99.2	99.9	100.0
16	Sandstone Scr	0.0	...	...	0.2	1.4	4.2	30.9	63.2	82.9	86.4	...	98.3	....	100.0
17	Aurora	0.5	...	...	0.9	1.1	1.6	12.3	35.5	55.7	61.3	...	97.8	....	100.0
18	Joliet	18.3	19.3	19.5	20.2	21.1	22.2	32.5	63.7	94.8	96.9	99.5	99.7	99.9	100.0
19	Moline	0.0	0.0	0.1	0.2	1.6	4.3	35.2	67.1	85.8	88.6	96.9	98.3	99.4	100.0
20	Urbana	3.5	6.7	8.6	17.0	39.1	52.4	74.4	79.6	83.5	84.5	92.3	95.3	98.3	100.0
21	East St. Louis	trace	0.4	0.7	1.8	5.9	9.9	39.5	55.1	79.4	82.6	95.9	97.8	99.4	100.0
22	Waukegan	0.0	0.1	0.2	0.8	8.8	12.1	38.4	61.7	82.0	85.8	98.0	99.0	100.0	100.0
23	Paris	1.2	1.7	2.1	4.3	12.9	16.9	64.4	91.4	97.6	98.1	99.1	99.4	99.8	100.0
24	Taylorville	4.0	7.1	9.4	22.1	55.6	75.4	99.5	99.8	99.9	99.9	99.9	100.0	100.0	100.0
25	Cairo	0.3	0.4	0.5	0.8	2.5	3.9	24.9	62.9	81.9	84.8	95.7	98.1	99.8	100.0
26	Hannibal, Mo	0.2	0.3	0.1	0.9	3.9	7.4	33.2	58.5	76.9	80.4	93.3	96.1	99.3	100.0
27	Beardstown	4.4	6.6	7.6	13.2	35.3	47.4	97.0	99.6	99.9	99.9	99.9	100.0	100.0	100.0
28	Mt. Carmel	2.4	2.6	2.7	3.9	18.8	34.2	98.7	99.8	99.9	99.9	99.9	100.0	100.0	100.0
29	La Salle	3.2	4.2	4.5	6.8	16.5	26.0	87.6	97.7	99.1	99.3	99.7	99.8	99.9	100.0
30	Peoria	2.7	3.1	3.5	6.0	14.5	17.8	47.8	71.9	88.2	90.3	96.7	98.0	99.0	100.0
31	Peoria	1.9	2.3	2.4	2.6	3.0	4.0	20.0	59.5	83.6	86.5	96.2	97.9	99.3	100.0
32	Peoria	0.6	0.8	0.9	2.0	9.2	13.3	68.7	89.1	96.0	97.1	99.0	99.4	99.7	100.0

Each sand was tested for cleanness, gradation of size of grains (sieve analysis), specific gravity, voids, and weight. The approximate mineralogical composition and comparative sharpness were also determined. Tests for tensile strength were made on mortars made of each of the sands. In making all of these tests especial care was taken to eliminate the personal factor from the results.

Tensile Strength.—Tensile tests were made on mortars composed of Portland cement and the various sands mixed in the proportions of 1:3 by weight. The cement used was a standard brand bought on the open market. Two bags of cement purchased at the same time were used. Tests of the neat cement from each bag gave the same results, within the limits of observation, as follows: Percentage passing the Nos. 100 and 200 sieves, 96 and 80 respectively; percentage of water for normal plasticity 21.0; tensile strength 612, 772, and 785 lbs. per square inch at 7, 28, and 90 days, respectively. The sand mortars were mixed with 9 per cent of water, corresponding to the 21 per cent above, as required by the "Specifications for the Uniform Tests of Cements" with standard sand. This amount of water was sufficient for the clean, hard sands, but with dirty or soft sands the mortars were quite dry. However, as the utmost care was taken in molding the briquettes, it is not believed that this has any considerable effect on the results, especially at the ages of 28 and 90 days. The mortars were mixed in batches sufficient for six briquettes. Eighteen briquettes were made from each sand. Two briquettes from each batch, or six in all from each sand, were tested at the ages of 7, 28, and 90 days. The entire series of briquettes were molded as nearly at the same time as practicable, and all other precautions were taken to maintain uniform conditions throughout the mixing and molding. After being molded the briquettes were kept in moist air for about 24 hours and then stored in water maintained at about 70° F. in tanks so arranged that the water was gradually changed. Upon reaching the required age the briquettes were removed from the water and immediately broken in a Riehle automatic cement testing machine, applying the load at the rate of 600 lbs. per minute.

The detailed results of these tests are given in Table I. It will be noted that these results are quite uniform, *i. e.*, the variation in the strength of the briquettes of the same sand at the same age is small. The sands were ranked in the order of the tensile strength of the mortars at 90 days. This ranking is shown in Table IV.

Cleanness.—The sands were tested for cleanness as follows: 1,000 grams of the sand were thoroughly agitated in about 1 gal. of water. The mixture was then allowed to settle for about one minute, experience showing that this allowed sufficient time for the finest sand to settle. The dirty water was then siphoned off with a ¼-in. rubber tube, care being taken that none of the sand was carried over with the water. This washing process was repeated until the water showed no discoloration. It was usually found necessary to "scrub" the dirtier sands between the hands to entirely remove the coating of clay from the grains. The sand was then transferred to a pan and as much water as possible drawn off by jarring the sand until the water flushed to the surface, and then removing the water with a pipette. The sand was then re-dried over the steam coil and weighed. The loss in weight due to the washing was taken as the amount of suspended matter.

The time required to accomplish this washing varied from about 15 minutes for a fairly clean sand to about 2½ hours for the dirtiest, the average being about one hour. The results of these tests are given in Tables II and IV.

Sieve Analysis.—The following standard sieves were used: Nos. 5, 8, 10, 16, 20, 30, 40, 60, 74, 100, 150, and 200. In some of the earlier tests sieves Nos. 5, 10, 15, 150, and 200 were omitted. The sieves were nested in order of size, the largest at the top. The washed sand was then placed on the top sieve and the whole shaken for 10 minutes on a "Per Se Sieve Agitator" driven by power at 100 r. p. m. After shaking, the sand retained on each sieve was weighed and from these weights the percentage passing each sieve was calculated. In making these calculations the amount of suspended matter, as determined by the cleanness test, was included with the material passing the finest sieve. The results of these tests are given in Table II.

Specific Gravity.—The specific gravity of the sands was determined by means of a Schumann specific gravity flask. This flask consists of a bulb or bottle into the neck of which a graduated stem fits with a ground joint. The bulb was filled with water and the height of the column of water in the steam read from the graduations. Fifty grams of sand were admitted, care being taken to permit the escape of the air, and the new height of the water column read. The difference in the two readings gave the volume of water displaced. Another 50 grams of sand were immediately admitted to secure a double determination as well as a check on the observations. The specific gravity was compiled from these data, the results being shown in Tables III and IV.

Voids.—After a careful investigation of the various methods of determining the percentage of voids the following method was used. About 20 c. c. of sand were placed in a graduated cylinder which was about 2 ins. in diameter and had a capacity of about 500 c. c. The sand was compacted by striking the cylinder lightly on a pad composed of eight thicknesses of heavy cotton flannel. Twelve blows were given at a rate of about two per second with a fall of about 1 in., care being taken that the blow was not hard enough to cause the sand to bounce, precautions also being taken to prevent the sand from being thrown from side to side. Successive increments of sand were added in this manner until the cylinder was filled. The difference between the weight of the empty cylinder and that of the cylinder full of sand gave the weight of 500 c. c. of dry compacted sand. Knowing the specific gravity of the sand the percentage of voids was computed from the equation

$$V = 100\frac{S-W}{S},$$

in which V is the voids expressed as a percentage of the total volume, W is the weight of the sand, and S is the product of the volume of the sand, its specific gravity, and the weight of a unit volume of water, *i. e., S* is the weight of an equal volume of sand containing no voids. In this case *S* equals 500 multiplied by the specific gravity.

The results obtained by this method are quite uniform, the maximum variation from the mean of five determinations of the weight of 500 c. c. of the same sand (made by the same observer) being only 7.5 grams, or about 0.8 per cent. The maximum variation from the mean of four sets of determinations on the same sands by four different observers was only 1.5 per cent. The results of these tests are given in Tables III and IV.

Weight.—The weight per cubic foot of each sand was computed from the weight of 500 c. c. and is given in Tables III and IV. These results average about 3 per cent higher than the results obtained by other observers on similar sands; but it is believed that this is due to a difference in the method of compacting the sands. It is also thought that the results given here are more nearly correct for the dry sands after being transported some distance in wagons or cars.

DISCUSSION OF TESTS.

Although this series of tests includes only a small proportion of the different sands used in Illinois, it is believed that a sufficient number of samples, well distributed geographically, have been tested to serve as an index to the mortar-making qualities of the sands available in various sections of the Illinois, and at the same time afford some information relative to the mortar-making qualities and other characteristics of sands in general. Following is a brief discussion of the results of this series of tests, together with the conclusions which may be drawn.

Mineralogical Composition.—The mineralogical composition of a sand is the fundamental factor in its mortar-making qualities, since not only its durability and hence the durability of the mortar but the size and gradation of the grains, the nature of the grain surfaces, the strength of the grains themselves, and all the other factors which affect the strength of the mortar are more or less directly dependent on the nature of the component materials of the sand.

The samples tested indicate that the majority of Illinois sands are composed of satisfactory materials. Quartz is the principal component, forming the bulk of the medium-sized grains, and in most cases more than half of the entire volume of the sand. Limestone is the next in importance, while the other rocks form comparatively a small proportion of the whole.

Specific Gravity.—The specific gravity of a sand affords but little information relative to its mortar-making qualities, its principal value being as a factor in certain computations. Quartz has a specific gravity of about 2.65; and the nearer the specific gravity of a sand approaches this value the greater is the content of silicious material. A higher value indicates considerable quantities of materials other than quartz, which are likely to be hard and durable; while a lower value usually indicates the presence of soft, unsatisfactory material, or of considerable quantities of clay and loam or other foreign matter. The specific gravities of the samples tested ranged from 2.60 to 2.75.

TABLE IV.—SUMMARY OF RESULTS OF TESTS TO DETERMINE THE CHARACTERISTICS AND MORTAR MAKING QUALITIES OF VARIOUS SANDS.

Sand. No.	Sand. Name.	Kind.	Tensile strength of 1:3 mortar at 90 days. Lbs. per sq. in.	Tensile strength of 1:3 mortar at 90 days. In per cent of the strength of Standard sand mortar.	In per cent of the strength of Neat cement.	Rank according to tensile strength.	Voids, pct.	Weight per cu. ft., lbs.	Suspended matter, pct.	Specific gravity.
00	Neat cement	Portland	785	213.4	100.0	..	...	....	...	....
0	Ottawa standard	20-30	368	100.0	46.9	7	34.6	108.7	0.0	2.660
1	Chicago K. I. Co	Lake	205	55.7	26.1	27	36.2	105.8	0.3	2.655
2	Chicago Z.-R. Co	Lake	311	84.5	39.6	11	34.4	108.8	0.3	2.655
3	Chicago K. I. Co	Lake	314	85.4	40.0	10	31.6	115.1	0.3	2.695
4	Limestone screenings	Joliet	485	131.8	61.8	1	28.5	122.8	0.0	2.750
5	Elgin, Hammond pit	Bank	464	126.0	59.1	2	31.9	115.7	1.0	2.720
6	Elgin, Stimpson pit	Bank	410	111.5	52.2	5	31.6	114.4	0.9	2.680
7	Bloomington	Bank	286	77.8	36.4	17	30.5	113.8	8.0	2.625
8	Rockford	Bank	386	104.9	49.2	6	32.0	113.0	0.6	2.665
9	Lincoln	Bank	303	82.4	38.6	14	32.4	111.9	1.1	2.650
10	Alton	River	269	73.2	34.3	20	31.5	112.5	0.3	2.630
11	Alton	River	280	76.2	35.7	18	31.0	114.5	0.2	2.660
12	Covington, Ind	Bank	310	84.4	39.5	12	33.5	110.4	1.5	2.660
13	Gladstone	Bank	241	65.5	30.7	25	35.8	106.3	0.2	2.655
14	Decatur	Bank	428	116.4	54.5	4	30.3	115.8	2.5	2.660
15	Freeport	Bank	317	86.2	40.4	9	33.0	113.9	1.3	2.720
16	Sandstone screenings	Freeport	269	73.2	34.3	20	33.0	112.2	0.0	2.705
17	Aurora	Bank	454	123.3	57.8	3	34.2	111.7	0.5	2.720
18	Joliet	Bank	159	43.3	20.3	31	39.7	100.2	18.3	2.660
19	Moline	River	257	69.8	32.7	24	32.3	112.0	0.0	2.650
20	Urbana	Bank	330	89.7	42.0	8	35.0	107.5	3.5	2.650
21	East St. Louis	River	258	70.2	32.8	23	30.1	115.8	Trace	2.650
22	Waukegan	Lake	296	80.5	37.7	16	32.7	112.5	0.0	2.680
23	Paris	Bank	259	70.4	33.0	22	35.2	108.3	1.2	2.665
24	Taylorville	Bank	211	57.4	26.9	26	39.4	99.7	4.0	2.635
25	Cairo	River	298	81.0	38.0	15	34.7	107.8	0.3	2.645
26	Hannibal, Mo	River	279	75.8	35.5	19	31.3	115.0	0.2	2.680
27	Beardstown	Bank	140	38.1	17.9	32	37.9	101.0	4.4	2.605
28	Mt. Carmel	River	170	46.2	21.7	30	36.5	104.4	2.4	2.635
29	La Salle	Bank	194	52.7	24.7	28	35.5	106.7	3.2	2.650
30	Peoria	Bank	262	71.2	33.4	21	32.9	111.9	2.7	2.665
31	Peoria	Bank	306	83.2	39.0	13	36.9	103.9	1.9	2.640
32	Peoria	River	178	48.4	22.7	29	34.6	108.7	0.6	2.660

TABLE III.—RESULTS OF TESTS TO DETERMINE THE SPECIFIC GRAVITY, VOIDS AND WEIGHT OF VARIOUS SANDS.

Sand. No.	Sand. Name.	D, water displaced by 100 g. of sand, cu. cm.	Specific gravity, $\frac{100}{D}$	S, 500x specific gravity.	W, weight of 500 c.c. of sand, grams.	Voids, $\frac{S-W}{S}$, per cent.	Weight per cu. ft., lbs.
0	Standard	37.57	2.660	1330.0	870.0	34.6	108.7
1	Chicago, K. I. Co	37.62	2.655	1327.5	847.0	36.2	105.8
2	Chicago, Z.-R. Co	37.65	2.655	1327.5	871.0	34.4	108.8
3	Chicago, K. I. Co	37.13	2.695	1347.5	922.0	31.6	115.1
4	Limestone screenings	36.40	2.750	1375.0	983.0	28.5	122.8
5	Elgin	36.69	2.720	1360.0	926.0	31.9	115.7
6	Elgin	37.27	2.680	1340.0	916.0	31.6	114.4
7	Bloomington	38.10	2.625	1312.5	911.5	30.5	113.8
8	Rockford	37.51	2.665	1332.5	905.0	32.0	113.0
9	Lincoln	37.72	2.650	1325.0	895.5	32.4	111.9
10	Alton	38.03	2.630	1315.0	901.0	31.5	112.5
11	Alton	37.57	2.660	1330.0	917.0	31.0	114.5
12	Covington, Ind	37.60	2.660	1330.0	884.5	33.5	110.4
13	Gladstone	37.65	2.655	1327.5	852.0	35.8	106.3
14	Decatur	37.60	2.660	1330.0	927.5	30.3	115.8
15	Freeport	36.80	2.720	1360.0	912.0	33.0	113.9
16	Sandstone screenings	36.97	2.705	1352.5	899.0	33.0	112.2
17	Aurora	36.75	2.720	1360.0	894.0	34.2	111.7
18	Joliet	37.60	2.660	1330.0	802.0	39.7	100.2
19	Moline	37.77	2.650	1325.0	897.0	32.3	112.0
20	Urbana	37.75	2.650	1325.0	861.0	35.0	107.5
21	East St. Louis	37.73	2.650	1325.0	927.0	30.1	115.8
22	Waukegan	37.25	2.680	1340.0	901.0	32.7	112.5
23	Paris	37.52	2.665	1332.5	867.0	35.2	108.3
24	Taylorville	37.88	2.635	1317.5	798.5	39.4	99.7
25	Cairo	37.79	2.645	1322.5	863.0	34.7	107.8
26	Hannibal, Mo	37.33	2.680	1340.0	921.0	31.3	115.0
27	Beardstown	38.40	2.605	1302.5	809.0	37.9	101.0
28	Mt. Carmel	37.90	2.635	1317.5	835.5	36.5	104.4
29	La Salle	37.70	2.650	1325.0	854.0	35.5	106.7
30	Peoria	37.50	2.665	1332.5	896.5	32.9	111.9
31	Peoria	37.80	2.640	1320.0	832.0	36.9	103.9
32	Peoria	37.61	2.660	1330.0	870.0	34.6	108.7

Sharpness.—Angularity or irregularity of the sand grains appears to exert no effect on the tensile strength of the mortar. In compression, the sharp sands may show a slight advantage due to the interlocking of the angular grains; but evidently such action is insignificant as compared with the resistance to displacement of the grains afforded by the bond between them due to the adhesion of the cement to their surfaces, hence the strongest mortars are invariably those in which the cement most readily adheres to the sand grains. The common requirement of specifications that sands for mortar and concrete shall be sharp is not only useless but may even be detrimental, and should be omitted.

Voids.—The percentage of voids in dry sand is a function of its compactness and the gradation of sizes in the sand grains, hence the effect of the voids on the strength of the mortar is included with that of the gradation of the sizes of the grains. The percentage of voids is valuable in determining the amount of cement necessary to give the densest mortar. The sands tested show voids varying from 28.5 to 39.4 per cent. The average sands contained 32 to 34 per cent when dry and well compacted.

Size and Gradation of Grains.—It has been demonstrated both by experiment and practice, that coarse sands will yield denser mortars than fine sands, and that the maximum density is obtained when the various sizes of grains are present in the proper proportions. Just what the proper proportions are, or in other words, what the ideal form of the sieve analysis curve is, has not as yet been determined. Experiments with materials for concrete indicate that for a mixture of cement, sand, and stone the sieve analysis curve should approximate a parabola, and by analogy it would seem that the same should be true of a mixture of cement and sand as well. Assuming that the parabola is the ideal line for the mixture, and remembering that the cement is very fine, it follows that the ideal line for the sand considered alone must lie *below* the parabola, and that it would be different for each proportion of cement used. But since the cement is very fine and forms a relatively large proportion in most mortars, the consequent variation in the ideal gradation for the sand considered alone will not be great; and hence it may be said that for the common proportions of cement and sand (1:1 to 1:4) the *flatter* the sieve analysis curve of the sand the denser will be the mortar made from it.

It has also been demonstrated that, other things being equal, the denser a mortar the greater is its strength; and hence the size and gradation of the grains is indirectly a factor in the strength of the mortar. Unfortunately, the fact that "other things must be equal" has been frequently overlooked with disappointing and sometimes disastrous results. It is only when the various sands are identical in mineralogical composition, condition of the grain surfaces, cleanness, etc., that the size and gradation of the grains becomes the controlling factor in the strength of the mortar, and even then it is only relative. Thus if two sands are exactly alike in all respects, except size and gradation of the grains, the better graded sand will yield a mortar of both greater density and greater strength; but this strength may be decidedly less than that of a mortar from a third sand which may be less perfectly graded than either of the two given sands but is superior in other characteristics.

Where strength is of the first importance in a mortar the only criterion is a direct test for strength; but after the absolute strength of the mortar from any one sand is determined, the sieve analysis may indicate whether the strength can be somewhat increased by improving the gradation of the sand. In general this improvement must be accomplished by judicious screening rather than by the addition of other material, since usually the added material will not be of the same general character as the given sand. Where a dense or impervious mortar is required, the size and gradation of the grains are of more importance; and if strength is secondary may control the choice of the sand. In this case the sieve analysis curves may be of considerable assistance in selecting the proper sand, or in indicating whether the desired results can be obtained either by combining several sands or by screening. In no case, however, should the direct test for strength be omitted, for a certain amount of strength is always required, and a good result for the sieve analysis is not sufficient evidence that the mortar will have sufficient strength.

The maximum size of grain permissible depends on the work for which the mortar is intended. For concrete the maximum size is now taken at ¼-in., but certain kinds of masonry, require a much smaller maximum size. The maximum size of grain fixes the limits of sieve analysis, and consequently two sands may differ greatly in average fineness, yet each one may be more perfectly graded and hence more suitable for some particular work than the other.

In general, the sands tested were quite fine and several of them were extremely so The bank sands showed the greatest difference in gradation. The river sands showed less difference in gradation than the bank sands, and, although the latter also showed the greatest difference in strength, this is probably due more to a greater difference in the other characteristics than to a difference in gradation.

Cleanness.—Foreign material in a sand may affect the strength of the mortar by retarding or preventing the hardening of the cement, by preventing the adhesion of the cement to the sand grains, and if present in sufficient quantities by simple "dilution" of the cement. Organic matter is the most common source of trouble, but inert clay and loam may prove deleterious under certain conditions. Experiments indicate that finely divided inert clay or loam may be present in an average sand to the extent of 10 to 15 per cent without appreciably affecting the strength of the mortar, provided the clay or loam does not adhere to the grains, while a very small quantity may seriously impair the strength of the mortar if it forms a coating around the sand grains.

Tensile Strength.—The tensile strength of the natural sand mortars varied from 140 to 464 lbs. per square inch at the age of 90 days. By comparing the tensile strengths with the sieve analysis curves it was noted that the coarser sands gave the mortars of greater strength. Only five of the natural sands gave a tensile strength greater than the Ottawa standard sand; and it will be noted that all of these are bank sands with rounded grains, comparatively well graded and containing considerable material other than quartz. The river sands showed less variation in strength than the bank sands, but this is to be expected since they differ less in composition, grading and cleanness. It may, therefore, be expected that river sands will form mortars of only moderate strength, and that there is likely to be little variation, between different sands; and that bank sands will furnish the mortars of greatest strength, but that the strength from different sands will vary through a wide range.

Table 1 shows that the mortars differ widely in the time required to attain approximately their maximum strength, and that the weak mortars invariably require less time than the strong mortars. This is explained by the fact that the strength of a mortar is largely dependent on the bond between the cement and sand. With poor sands the maximum bond is reached long before the cement has reached its full strength, while with the better sands the maximum bond is not attained until the cement has more completely hardened; hence more time is required. The 7, 14, and possibly 28-day tests are therefore not absolutely reliable for comparing the ultimate strength of different sand mortars. For such comparisons the longest time available should be used. The short time tests are, however, of value in determining how soon it is permissible to remove forms or to subject the material to limited use.

Crushed Stone Screenings.—Although the limestone screenings (No. 4) formed the strongest mortar, it must not be concluded that all screenings are superior to natural sands. This sample of screenings was of exceptional quality in every respect, while the natural sands tested were the average quality met in practice. The sandstone screenings (No. 16) gave only medium results. The results for the limestone and the sandstone screenings indicate that there may be as great a variation in the mortars made from screenings as in those made from natural sands. These two samples of screenings are not sufficient to warrant any general conclusions as to the relative merits of sands and screenings except that under favorable conditions of quality of stone and of crushing, screening, and handling, the screenings may compete favorably with good natural sands.

Proportioning.—These tests indicate that the variation in the mortar-making qualities of sands may be so great that a stronger mortar may be obtained from a lean mixture of one sand than from a much richer mixture of another sand, and since the cement is the most expensive part of the mortar a substantial saving in the cost of materials may be effected by a judicious choice of the sand.

The selection of the proper proportions for a mortar is essentially a problem in economics, whether the result to be attained is a mortar or a concrete of maximum strength, or one of the greatest density and imperviousness, or simply one of sufficient strength for the purpose in hand. The object in any case is to obtain the desired result with the minimum cost. If this fact is kept in mind it will become clear that it may really be true economy to spend what appears to be a considerable amount of time and money in investigating the available sands and determining the best combinations. Obviously, the saving to be effected varies with the magnitude of the work, and hence the bigger the job the more important it is that the materials be properly selected. If such investigations were generally made it would certainly result in reducing the careless and extravagant use of cement, the most costly material entering into mortar or concrete.

Water Works

Proposed Works for the Additional Water Supply of Toronto, Ontario.

The water supply for the city of Toronto is obtained from Lake Ontario through two submerged intakes sunk on the lake bottom, from each of which a steel pipe runs to a well on Toronto Island. After passing through the filtration plant the water is conveyed across the island in a steel pipe and under the bay by means of a tunnel which terminates at the John Street pumping station. From this station the water for the lower part of the city is delivered directly into the mains, while that for the upper part of the city is delivered to the high level station, where it is repumped. The Rose Hill reservoir serves as a storage for the lower district, the John Street pumps being directly connected to it.

Owing to the way in which the ground rises northward from the bay, the city has been divided into three districts (exclusive of the recently annexed North Toronto) for distribution purposes, the division lines between these districts being roughly by parallel lines along College Street and Dupont Street These districts are called the low level middle level, and high level respectively.

In 1911 there was only one intake from the island into the lake, and through troubles of various kinds, this intake broke near the shore causing very great inconvenience and trouble The city council then appointed a commission of four engineers to report upon the whole matter of Toronto's water supply This commission presented a report in February, 1912, covering a number of matters and recommending, amongst other things, a duplicate intake off the island, which was later installed, and further, that an entirely new system with intake off Scarborough plateau be constructed. The positon suggested for the latter is shown in Fig. 1.

After this commission's report was presented, several changes took place in the organization of the city engineer's department, and on the resignation of Mr. Rust, Mr R. C. Harris was appointed commissioner of works in 1912. After a study of the Scarborough scheme he felt that he could not be definitely committed to it, and as there was need of something being done as early as possible, he recommended that a money by law relating to water supply be submitted to the taxpayers in the very widest terms so that on Jan 1, 1913, the citizens voted $1,677,000 "for additions and extensions to the water works pumping and distributing plant."

The money having been voted by the people, Mr Harris felt that it would be safer to have the whole matter gone into by local men who have been in intimate touch with water works matters for some time. Prof. Robert W. Angus of the University of Toronto was appointed as Consulting Engineer on the work about the end of April, 1913, and had with him Mr. James Barr, of the works department, a man with an intimate knowledge of the city's plants. It is from information furnished by Prof Angus that the present article has been prepared.

After a most careful study of the report of the commission of 1912, Mr. Barr and Prof. Angus found it impossible to indorse the Scarborough scheme, for reasons set out elsewhere. A second plan, called the Victoria Park project, was then worked out and is described here in some detail Much of this description has been taken from the report of Commissioner Harris to the council in December 1913. The article relates to the location of the intake as well as to its proposed design

INTAKE LOCATION.

In selecting the point from which a water supply is to be drawn the most important controlling factors are (1) Reliability of supply; (2) Comparative purity of the water, and (3) The location must admit of convenient distribution of the water throughout the city Some one of these points may have to be selected as the controlling factor. Thus, there must always be a reliable supply, but so long as the water is quite safe and good, it may be necessary to allow the problem of distribution to be the determining factor in selecting the site.

There are three possible places at which such installation may be located. 1. At Toronto Island, adjacent to the present system. This site would naturally be suggested. 2. At some point west of Humber Bay. 3. At some point east of the island, which would mean at least as far east as the Woodbine racecourse. The relative merits of the three locations are as follows:

The Island Location.—Under many circumstances this would be the most natural to adopt, inasmuch as it is at the center of the city, and contiguous to the section where most water is used. Study shows that it would not be satisfactory.

The bed of the lake slopes off gradually to a depth of 23 ft. at about 1,700 ft. from shore and then falls with a grade of over 10 per cent, making the problem of constructing an intake difficult and uncertain.

It is essential to safety that the new intake be placed at some distance from the present ones to render improbable the failure of all at the same time.

As to purity, it is quite possible to treat the present supply so as to make it perfectly safe and very desirable. The location could not be rejected on this score.

As to distribution Toronto is increasing in size with very great rapidity. From 1890 to 1910 the population more than doubled, and the rate of growth since 1905 has been greater. This has naturally been accompanied by an increase in the extent of the city. At present, the distance from the easterly to the westerly boundaries is approximately ten miles, while from the bay front to the northerly limit of North Toronto approximates 6.4 miles.

The municipality will, doubtless, continue to expand to the north, east and west, with the probability that the greatest accretion of territory will be to the north.

As the breadth of the city increases, the problem of water distribution from a single source becomes more and more difficult, costly and unreliable, for if accident occurs at the source of supply, famine is imminent, and the city is left entirely without reserve. Toronto has outgrown the stage where such a condition is permissible, and must follow the admittedly prudent policy of large lake cities similarly placed, and increase the number of intakes as the area of the city increases, in order to ensure uninterrupted supply, and furnish additonal distribution centers.

Cleveland, Chicago and Milwaukee may be cited in this connection. The first-mentioned city, with a population in 1912 of about 600,000, and a water consumption of 61,000,000 imp. gals. daily, is placing a second intake at the present time; the same is true of Milwaukee, a city of about the same size as Toronto, while Chicago (population 2,250,000 in 1911) with an extreme length of 27 miles has six intakes and two more are in contemplation. In Chicago there are land tunnels connecting the suction sides of various pumping stations, thus, from the distribution standpoint, practically increasing the number of sources of supply.

It must be conceded that Toronto must increase the number of intakes in proportion to the expansion of the city, and that the intake tunnels must supply pumping stations at convenient distribution centers, unless there be some overruling condition which prevents. At present, one new intake situated near one of the present city boundaries is imperative, and before the lapse of many years a second may be required near the other boundary.

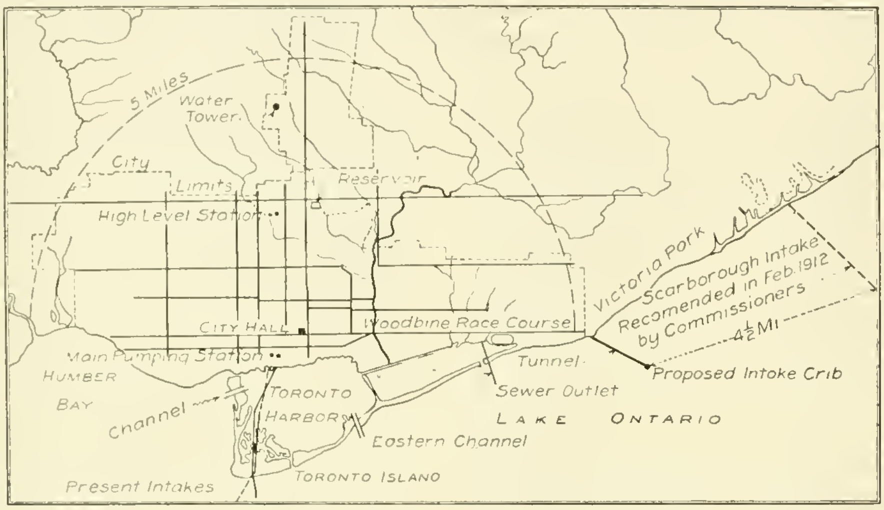


Fig. 1. General Map of Toronto Showing Location of Present, Rejected and Proposed Water Supply Intakes.

The question then resolves itself, into whether the new intake and works shall be placed at the east or west side of the city.

The Westerly Location.—Fig. 1. shows that this location would necessarily lie to the west of Humber Bay, and it is probable that in the future an intake may have to be constructed at this point. A study of its possibilities will be made.

As to purity of supply, it is well known that there is at present pollution around Humber Bay and to the west, due to the condition of Humber River water and other factors.

This cannot in any way be viewed as an insurmountable difficulty inasmuch as proper treatment of the water would render it quite potable.

As to reliability, there is little room for doubt that a safe intake could be constructed, although a survey would have to be made, along with lake borings, before a definite opinion could be offered.

There is, however, a very serious objection to this location in so far as distribution is concerned as it lies considerably to the west of the present city boundary, and would thus require much large piping to conduct the water to the point where it would be first used.

The western location, therefore, would not be desirable.

The Easterly Location. — There remains, then, only the location to the east of the island. The westerly limit for such would necessarily be at the Woodbine racecourse since the island ends at that point.

It is desirable to keep a safe distance from any large discharge of sewage. Having regard for the effluent from the Morley Avenue sewage disposal plant, and the westerly trend of the lake currents, we are forced to the east. On the other hand, the further the distance to the east the more undesirable becomes the location from a distribution standpoint. In order to get the best light possible on this extremely important phase of the question studies of the shore line and contours were made.

An examination of the short line shows that while the ground is fairly level, and only about 40 ft. above the lake at Victoria Park, it rises at a rapid rate to the east, until reaching the very high ground forming the Scarborough Plateau. Should a point be chosen to the east of Victoria Park, therefore, the pumping station would have to be on made ground at the water's edge.

The foregoing, coupled with the results of preliminary surveys on land and lake, and the very desirable positon of the park as a distributing center, caused the Victoria Park site to be selected tentatively. A complete study has confirmed this selection. As this location differs very materially from that chosen by the Board of Commissioners of 1912, arguments are presented which not only show the desirability of the park location intrinsically, but prove that it has very decided advantages over the Commissioners' location, which is some five miles to the east

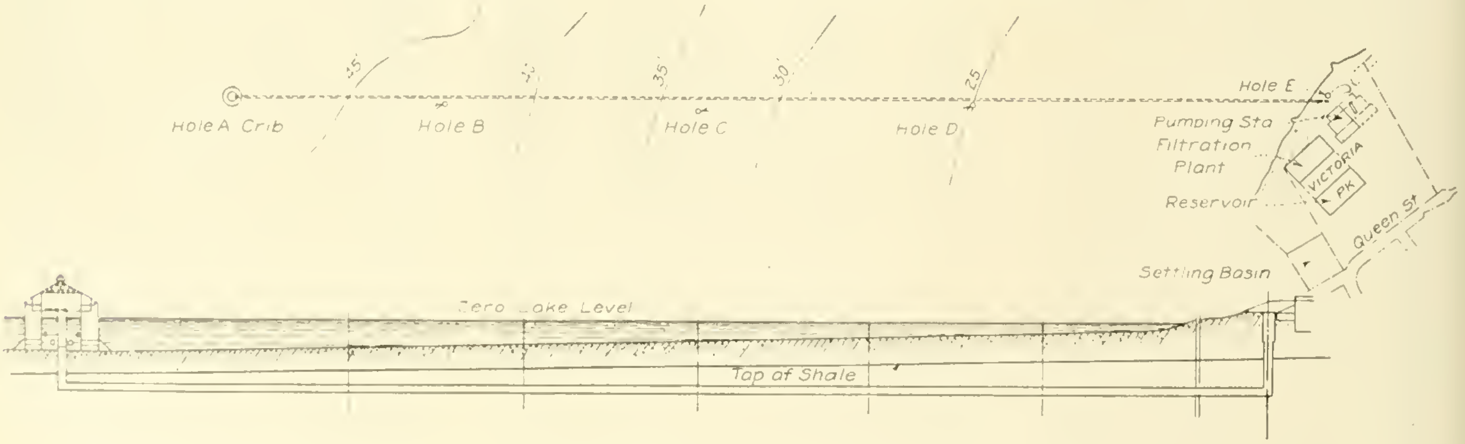


Fig. 2. Plan and Profile of Proposed Water Works Intake Tunnel at Toronto.

of Victoria Park on the Scarborough Plateau as shown in Fig. 1.

COMPARISON OF VICTORIA PARK AND SCARBOROUGH INTAKES.

Purity.—Comparing these two locations from the essentials of a good source of supply, the matter of purity will be first dealt with.

There is a false sense of security in the idea, that by going, say, seven miles from the Morley Avenue sewage disposal works, or any source of pollution, as the Commissioners proposed, pure potable water can be secured without after treatment. There can be no question, that in general, the further an intake is from any source of pollution, the purer will be the water, but this factor must only be allowed to exercise its proper weight, having regard for other influencing conditions, and the fact that any supply available for this city must be made safe by sewage treatment, water filtration and chlorination.

It is true that the rate of dilution of sewage in the lake increases as the distance from the outlet increases, so that up to a critical point there is a gain in increasing this distance, but beyond certain ranges the dilution is so great that a practical limit is set.

Again, it is well known that bodies of polluted water may move about the lake for miles without diffusion, and may in this way endanger, to some extent the supply to any intake within several miles of the city, if sewage purification and filtration be not adopted. In this connection it is helpful to refer to the report to the City Council by Dr. G. G. Nasmith on October 5, 1911. Dr. Nasmith says:

> At different times, sewage polluted water has been obtained at points as far apart as nine miles east of Toronto, five miles west of Toronto, and three miles beyond the Island.
>
> On July 26, after a strong westerly gale, samples were obtained two miles east of Dutch Church at Scarborough, and one mile from the shore, which showed sewage contamination.
>
> This was the most graphic demonstration of what a heavy gale could do in driving sewage-laden water long distances.
>
> From these analyses, and from the ocular demonstrations which we have had of these currents flowing in various directions with the wind, I am forced to the conclusion that currents of sewage polluted water may, at times, flow almost directly from the Eastern and Western Gaps to any intake that may be placed within a radius of several miles of the city.

Continuing his report, Dr. Nasmith speaks of the pollution due to storm water and says:

> From what I have seen this summer, and from the evidence I have laid before you, I have little doubt but that at times this polluted water will be carried to and beyond our present intake or any other intake that may be subsequently laid.
>
> I would sum up the report by saying that this investigation has proved, to my mind . . . that we cannot, with our sewerage system as it is being constructed, hope to keep all untreated sewage out of the lake all the time, and that, therefore, . . . we must to obtain pure water use both filtration and sterilization

In view of these statements of Dr. Nasmith, it is evident that there is no hope of obtaining pure water at all times, at any reasonable distance from Toronto, and, therefore, the matter of primary purity must not be allowed to unduly affect the location of the intake, as the water must be treated in any event. The improvements in the treatment of water have been so great that water taken either off Victoria Park or Scarborough Plateau can be made perfectly safe and satisfactory for drinking purposes, and, indeed, at either place the raw water would be quite pure during the greater part of the year

The report of the Commission as relating to the purity feature, may be again quoted

> These investigations also lead to the conclusion that there is no locality along the lake front, from Mimico to Scarborough Heights, which has an assured purity, and, further than that, it is demonstrated that waters 15 miles from shore are not free from contamination; hence there is no one locality within the range under consideration, at, or on, which you can locate a water intake with assurance that the water taken into it will be free from contamination; although at times healthful water could be secured at almost any chance location. And, therefore, we have no expectation of obtaining a water supply which must not be subjected to efficient artificial method of purification.

From the point of purity, therefore, the water opposite Victoria Park, if taken at a distance of about one mile from shore and at a depth of about 40 ft., would be perfectly satisfactory, and fully as desirable as that taken from further east. This location is two and one-half miles from the Morley Avenue outlet.

Another consideration which indicates the superiority of this location is the fact that, though the effluent from Morley Avenue sewage disposal works is at present treated so as to largely deprive it of its contaminating influence, the city will, by the time the Victoria Park project could be completed, have in operation at Morley Avenue a system of purification which will render this effluent innocuous.

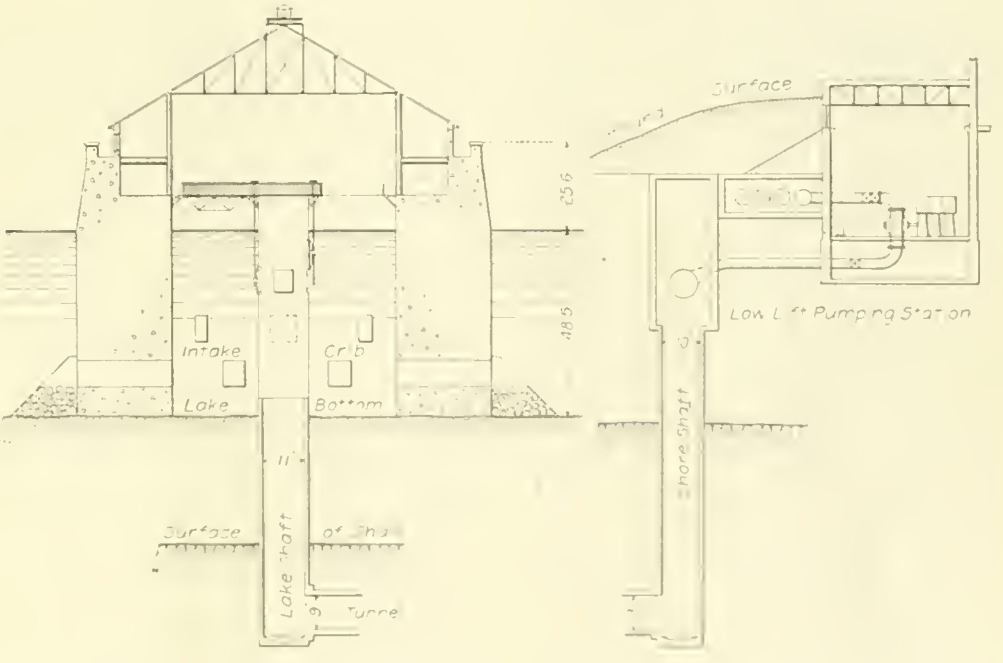


Fig. 3. Sectional Elevations of Proposed Intake Crib, Lake and Shore Shafts and Tunnel for Proposed Additional Water Supply for Toronto, Ontario.

Dr. Nasmith, under instructions from Dr. Hastings, has for six months past been conducting experiments with a view to arriving at the best solution of this particular problem, and as soon as a conclusion is reached, a recommendation will be made to the council to give effect thereto.

Turbidity.—The question of turbidity is one of much importance. In order to ascertain the conditions at the selected point, observations have been taken, whenever there was any indication of disturbance of the clay, since the middle of September, but it has only been possible to obtain samples on three days, because of the fact that the turbidity has not extended out to the proposed site of the intake crib. In each case the observations were made after fairly heavy storms, although these were not of so long duration as would occur at other seasons of the year

At the Victoria Park intake crib site the turbidity at 20 ft. and over, below the surface, did not exceed 2.0, while on one of these days the turbidity at the Island intake was 37, on another day it averaged 17 and on the third day it was 11. The proposed site is thus superior to the present intake locations in this respect.

Temperature.—Observations of temperature show that the water at the park site will be slightly warmer than at the island, the average difference being only 2° F.

Reliability.—As to reliability of supply the Victoria Park site furnishes an almost ideal location. The accompanying tunnel plan, Fig. 2, shows a series of soundings taken at this point, and indicates a gently sloping lake bottom, for a distance of two miles and over, on the line of the tunnel. At about 2,100 ft. from shore, the depth of water is 25 ft.; at 3,700 ft. it is about 33 ft., and at 6,400 ft. out it is nearly 49 ft. The average slope of the lake bed is thus less than 1 per cent., furnishing a spot sufficiently level to place an intake at any desired point. (All depths are referred to Zero lake level).

The results of a series of borings made at this location, indicate that at about 6,400 ft. off Victoria Park, in a depth of 49 ft. of water, shale was struck at 82 ft. depth. At about 5,200 ft. from shore, in a depth of 42 ft. of water, shale was struck at a depth of 72 ft., and at corresponding depths at other locations, the depth of the shale at the shore being 50 ft. These borings all show that it is quite possible to tunnel in shale, to an intake placed nearly one mile from shore, and in a depth of approximately 49 ft. of water, driving the tunnel in good shale, without going to a depth which would preclude compressed air working should it prove necessary. Figure 2 also indicates the profile of the lake bottom, and the line of the shale as found from the actual borings.

It is quite possible to construct at this site a tunnel and intake, which will, without doubt, ensure safety and reliability

Another advantage of this position is its convenience as a distributing point to the present city system The city is growing in all directions and Victoria Park will probably, in a few years, be surrounded by a populous district

A station at this point will be an admirable position for connection to the present system It is not too far removed from the sections where most of the water is used, and any main laid from the station will be used for distribution from the point where it leaves the park

In connection with the present project, additional reservoir capacity is not provided as it is deemed unnecessary. This will be discussed later

THE PROPOSED WORKS AT VICTORIA PARK

General.—In general it is proposed to build an intake crib off Victoria Park, about one mile (5,100 ft.) from shore, in 49 ft. of water. The crib will be approximately 110 ft. in diameter, constructed of steel and concrete, and surmounted by a house, the main part of the crib rising 19½ ft. above the lake. The house will carry a permanent light and the requisite fog signal apparatus, and include living rooms for men. The crib will contain the necessary ports for the admission of water, as shown in Fig. 3.

The inside of a crib will contain a vertical steel shaft, lined with cement or brickwork, running down to a proper depth below lake level, so as to connect with the tunnel 6,380 ft. long, running horizontally to the shore at Victoria Park. The internal diameter of the shaft will be 11 ft., that of the tunnel 9 ft., the latter to have its invert 105 ft. below mean lake level. At the shore, there will be a vertical shaft 10 ft. in diameter, rising above the normal lake level at this point. From the vertical shore shaft there will be two horizontal 7-ft. tunnels, one on each side and parallel with the shore. One of these tunnels will supply the suction side of the pumps in the station to be erected, the other will act as a reserve to be used in case the size of the station is increased later.

The pumping station will be located in Victoria Park, which, with a piece of land to the east, is of sufficient size to contain a station and filtration plant of a capacity of 120,000,000 imp. gals. per 24 hours, should such capacity ever be required. There will still be room for a fair-sized park. A protecting breakwater will be erected and the banks will be terraced.

The pumping station will have an initial capacity of 60,000,000 imp. gals. per 24 hours, but is being so designed that this may very easily be increased to 120,000,000 gals. if desired, after the removal of a single end wall. Since the water will have to be filtered, the station must be divided into two parts, viz., low-lift and high lift, the former taking the water from the tunnel shaft and delivering directly to the filters, while the latter will deliver the filtered water direct to the city mains. Each part will have a capacity of 60,000,000 imp. gals. per 24 hours. The low-lift pumps will work against a head of approximately 75 ft., while the approximate head on three of the high-lift pumps will be 220 ft., or 95 lbs. per square inch, and that on the fourth pump about 315 ft. or 137 lbs. per square inch. The exact pressures against which the low-lift pumps will have to work cannot be determined until the filters are designed.

The mechanical type of filter is recommended, and in connection with this it is proposed to install a reservoir with a capacity of 2,500,000 imp. gals. and to provide space for a settling basin should the make of filter require it.

From the station, four 36-in. pipes will deliver water to four new 42-in. mains which will join the present distribution system. This plan contemplates:

1. A system which will supply an additional quantity of water equal to that now being used during maximum draft, thus practically duplicating the present plant.

Therefore, in the event of accident at any time to the sewage purification plant, or either filtration or pumping plants, the use of either plant could be discontinued, and the safety of the supply still maintained. By reason of the proposed plant being a duplicate in capacity of the existing plant, with provision for future increase, the continuity of the supply is assured.

2. A system which will operate in exact harmony with the existing plant, thus enabling both to work in conjunction.

3. An arrangement which can be doubled in capacity without doing unnecessary work.

4. A supply which is unfailing, in so far as human agency can make it. It will not be deleteriously affected by storms, ice, shifting sand, etc.

5. A supply of water, which is as pure as it is possible to obtain in the vicinity of Toronto, and which after treatment will be perfectly safe and satisfactory.

6. An improved service throughout the entire city.

7. An attractive park on the waterfront.

DETAILS OF WATER WORKS EXTENSION.

Intake Crib.—The intake crib is, in a sense, the most important, and, in some ways, the most difficult part of the work. On account of the nature of the elements to which it is exposed, the design should be worked out from the experience of others. Cribs have been constructed in the lakes by various other cities; at Buffalo the crib is 110 ft. diameter in 18 ft. of water; the Cleveland crib is nearly five miles from shore in 49 ft. of water, and is 100 ft. in diameter. Chicago has six cribs of varied construction, some with surrounding breakwaters, and the latest designed without. Of the latter, the Harrison crib is 112 ft. diameter and is in 34 ft. of water, while the new Dunne crib is 111 ft. diameter, and in 33 ft. of water. The Milwaukee crib is under 60 ft. diameter, and is in 25 ft. of water, but from a study of conditions, it appears to be inadvisable to adopt so small a size. From a study of the experience of these cities, it is concluded that the Toronto crib should be 110 ft. diameter.

The depth of water in which the crib can be set depends upon the method of placing it, and the forces to which it will therefore be subjected. This crib, made of a steel shell, will be constructed in the harbor and floated to the desired location and sunk. On account of the depth of water in the bay, the shell could not be floated to a greater depth than 17 ft. It is clear that the success of such an undertaking will depend, to a very large extent, upon the relative amounts of the shell above and below the water surface when floating, and as the depth below is, in this case small, the total height of the structure is limited. So far as known, no city in America has installed a crib under such conditons as those existing at Toronto, in water deeper than 50 ft., the Cleveland intake being the deepest.

Since the cost of this part of the work will be great, and hence any unsuccessful experiment would be very serious, it is not advisable to attempt anything that has not been done successfully elsewhere, so that the depth of water in which this crib will be placed is 49 ft. The water entering such a crib will be quite as good for drinking purposes as if taken at much greater depth.

The intake crib will consist of two concentric steel cylinders, set on end and firmly braced together, so as to be rigid and stiff, the outer cylinder being 110 ft. diameter, and the inner 60 ft. diameter. Plates set radially between these shells will divide the annular space into compartments, which will be constructed with bottoms so as to make them watertight. The compartments will then be sufficiently filled with concrete to make the draft of the structure 17 ft., after which it will be towed into the desired position and sunk, the lake bottom having been previously levelled and otherwise prepared for its support. After sinking, the annular space will be filled with concrete, and the structure built up above the water to provide living quarters for men, with lights and signals for mariners.

Properly constructed ports in the crib will admit water to the inner 60 ft. well; these ports will be controlled by gates which may be closed at pleasure, but the water will never be pumped from the inside of this well. An arrangement has been provided, however, so that it will be quite possible at any time to pump all of the water out of the tunnel for cleaning and other purposes, should this be desired.

Intake Shaft.—Through the center of the intake crib, a vertical steel shell will be sunk to a depth to meet the tunnel. This shell will rise inside the crib above lake level, and will have proper openings for the admission of water to its interior; these ports will have gates that may be closed when desired. The material will be excavated from the inside of this shaft, which will be lined with brickwork to finish 11 ft. inside diameter.

Tunnel.—A tunnel will be driven between the shore and intake shafts having an inside diameter of 9 ft. with a brick or concrete lining. As it is possible that compressed air will be necessary in the driving of this tunnel, and in order to make it perfectly safe for workmen, the invert will be not over 105 ft. below the lake level. The length of the tunnel will be about 6,380 ft.

Shore Shaft.—This shaft will be of similar dimensions to that at the intake, but with a diameter of 10 ft. inside the brick or concrete lining; will be closed at the top, and at 13½ ft. below Zero lake level will have two 7-ft. diameter horizontal branches, to which the suction pipes of the pumps will be directly connected. The cover on the shore shaft will be high enough to prevent damage from surges in the tunnel, these being very small, owing to the low tunnel velocity of only 1.71 ft. per second when delivering 60,000,000 imp. gals. per 24 hours.

Capacity of Tunnel and Shafts.—With a draft of 100,000,000 imp. gals. per 24 hours the loss of head in the tunnel and shafts will be under 2.5 ft., so that the capacity will be much above 100,000,000 gals. if it is desired to increase the demand above this amount at any time.

PUMPING STATION AND FILTRATION PLANT.

The general scheme of the station has been described already. There are two parts which will be described separately.

Low Lift Station.—The purpose of the pumps in this station will be to take water from the tunnel shaft and deliver it to the filters. They will be driven by steam and have a total capacity of 60,000,000 imp. gals. per 24 hours. The lift at zero lake level may be approximately 75 ft., depending upon the make of filters used. In order to procure the best use of the tunnel, the floor of this station should be at lake level. An excavation will be made and the station built with watertight walls and basement. In order to insure safety, there will be two 48 in. delivery pipes to the filter plant, each conduit carrying away the water from two pumps.

High Lift Station.—This station will also contain steam driven pumps, having total capacity of 60,000,000 imp. gals. per day, three having a total lift of approximately 220 ft., and the fourth with a lift of about 315 ft. The precise lift is yet to be determined. The floor level of this station will be at the ground level of Victoria Park.

Filtered water will be delivered to this station through a single 7-ft. pipe, the suction of each pumping unit being connected directly thereto. There will be four separate discharge pipes, each 36 ins. in diameter, one from each pump, and connected to the distribution system.

Boiler Room, Coal, Storage, Etc.—The boiler room will be located between the engine rooms. This arrangement brings the boilers close to all pumps, thus giving good steam connections, and also precludes any interference whatever between the boiler and engine foundations and the water pipes. This latter matter is of the very greatest importance. Much attention has been given to the security of the system, and the prevention of damage to the pipes from any cause. It is essential that they must always be easily accessible for examination and repairs.

The boilers will be equipped with mechanical stokers, and the boiler room will be furnished with coal and ash handling machinery, so as to reduce the cost of operating the plant to a minimum.

Coal will be conveyed to the plant in railroad cars drawn by electric motor, along tracks laid to York Station, a distance of 1½ miles.

The coal bunkers will have a storage capacity equivalent to one week's supply.

RESERVOIR.

The matter of an additional reservoir is one requiring careful study. There is no convenient location for a reservoir in the eastern part of the city, and it would have to be located far beyond the city limits if placed in this direction.

An additional reservoir for the city of Toronto is by no means essential, and not at all requisite. It is desirable that there should be some water storage to provide for fluctuations in demand from time to time. This storage has already been adequately provided for in the Rose Hill reservoir, which has ample capacity for all the present and proposed pumping equipment, and will readily suffice for the needs of Toronto for some time to come.

An examination of the country has shown that to secure suitable land for an additional reservoir it would be necessary to go so far out that much of its effect would be lost. Such a plan cannot be recommended.

The present Rose Hill reservoir has a capacity of 39,250,000 imp. gals., which corresponds to about 20 hours' supply at the present average daily pumpage of approximately 50,000,000 imp. gals., and is quite sufficient to compensate the ordinary fluctuations of discharge, due to fire, excessive heat or cold, and other causes.

It will be interesting in this connection to give the experience of some of the larger American cities, using lake water. Buffalo, with an average daily consumption of 117,000,000 imp. gals., has a single reservoir holding 98,000,000 gals., or 20 hours' supply. Detroit, with an average daily consumption of nearly 88,000,000 imp. gals., has no reservoir. Chicago does not possess a reservoir. In Cleveland the average daily consumption is 63,000,000 imp. gals., and the city is divided up into three districts for distribution purposes, there being a reservoir for each district, the largest having a capacity of 67,000,000 imp. gals. Milwaukee has a present consumption of about 79,000,000 imp. gals., and is now increasing its pumping capacity, but there is no intention of increasing the capacity of the reservoir, which holds only 17,000,000 imp. gals., or about six hours' supply.

The foregoing examples show that Toronto has now a much larger reservoir capacity than many other of the larger cities, the size of reservoirs being evidently largely influenced by local conditions. The experience of the cities quoted confirms the conclusion otherwise reached, that there is no necessity at present to increase the reservoir capacity for this city.

DISTRIBUTION SYSTEM.

From the pump house there will be four 36-in. pipes, with Venturi meters, running a short distance from the station, after which they will be enlarged for distribution purposes, and will feed four 42-in. mains, all of these pipes being cross-connected and controlled by valves in such a way as to make a perfectly flexible arrangement. The details of the proposed extension of the distribution system are chiefly of local interest and are not here given. The need of new mains was determined by, and the design of these mains has been based upon, extensive pitometer surveys made for that purpose.

The estimated cost of the project recommended is as follows:

ESTIMATED COST OF PROJECT AS RECOMMENDED.

Lake crib—complete and placed	$ 400,000
Lake shaft, tunnel and shore shaft	800,000
Site and buildings	500,000
Pumping equipment	380,000
Boilers and stokers	80,000
Coal and ash handling machinery	30,000
Tracks and overhead equipment from York to pumping station	35,000
Motor and equipment for handling cars	5,000
Electric lighting unit at station	2,000
Twenty-six miles of distribution main	2,040,000
Specials and valves	60,000
Filtration plant	600,000
Levelling, grading, etc	5,000
	$4,937,000
Add 10 per cent for engineering and contingencies	493,700
	$5,430,700
Six miles of distribution main High level pumping station to West Toronto, for part of which contracts have been awarded	$ 353,000
Additions to High Level Pumping Station and provision of steam reserve sufficient for ultimate output of this station, for part of which contracts have been awarded	250,000
Total	$6,033,700
Recapitulation—	
Expenditure authorized by taxpayers	$6,677,000
Estimated cost of propject as proposed	6,033,700
Balance to meet debenture discount and financing	$ 643,300

The project was submitted by Commissioner Harris of the city and the provincial health departments, both of whom approved it on the understanding that the sewage effluent from the Morley Ave. works should be properly purified as required by the Public Health Act. As this treatment of the sewage is required by law there should be no trouble from this source and the matter is now receivng the attention of the authorities.

Dam Foundations.

To the Editors:—The writer feels constrained to make a few remarks concerning dam foundations and the effect of the character of the foundation on the type of dam to be selected. It seems generally to be considered necessary, when trying to draw attention to any important point, to emphasize that point to the exclusion of other perhaps equally important points.

Mr. Edward Godfrey in Engineering and Contracting, Feb. 11, 1911, says, "It is a crime to design a dam without considering upward pressure." I think in a general way that statement is correct, with emphasis on the word "considering," but do not think that means that in every single case such upward pressure must be included as one of the forces acting on the dam. I think there are cases where a consideration of upward pressure in connection with *thorough* examination of the foundation leads to correct conclusion that upward pressure of water will never act on the base of that proposed dam.

I agree with Mr. Godfrey that upward pressure has contributed largely to many dam failures. He also states in the discussion above referred to that "no dam ever failed that was heavy enough and wide enough at the base to be stable under the pressure of water on the wetted side and underneath it, regardless of what it was founded on." Perhaps that is so, but does Mr. Godfrey mean to say that he would use no other type of dam anywhere, for instance, that he would use such a massive type of dam "regardless of what it was founded on" for such a foundation as was under the portion that failed of the Stony River Dam? I do not think he would actually build any dam "regardless of what it was founded on" for such neglect of an important phase of engineering design and construction seems entirely out of harmony with his character as indicated by most of his contributions to engineering literature which have come to my notice.

I do not know how many dams have been built as heavy as indicated in the above quotation. If a triangular cross-section with vertical up stream face be considered and the upward pressure be taken as equal to full hydrostatic pressure at the heel, decreasing uniformly to zero at the toe and the weight of masonry be taken as 150 lbs. per cubic foot, I make out that the center of pressure would remain within the middle third when the thickness at base equaled 85 per cent. *h* if *h* is depth of water and height of dam. I also make out that in the same case the horizontal water pressure would equal 85 per cent of the effective weight of the dam, so that even for that thickness, I think it would be worth while to pay some attention to the foundation, i. e., to see that it was better than it proved to be at Austin dam, Pennsylvania, and also Austin dam, Texas. Perhaps in those cases an upward pressure greater than zero was acting between some of the rock layers beneath the toe of dam at time of failure.

It seems to me that it is better engineering to make a thorough examination of the foundation and, where possible, either improve it by grouting or other methods, or prevent under passage of water by a cut-off wall *that will cut off* the flow.

I do not believe in condemning a particular type of dam, or any other structure, merely because there are instances of failure recorded against that type. If so, where shall we stop and what structures can we build? If the foundation of a dam was faulty and it failed, why not charge at least a part of the failure to the faulty foundation, since in many cases it would have been altogether possible and feasible to have foreseen the probable failure and prevented it by an effective cut-off wall or other method, as, for instance, Austin, Pa., dam, Stony River dam, Hauser Lake steel dam, Pittsfield dam, and others. That does not mean that man was in no way responsible, for he should know enough about foundation conditions to prevent most of such failures.

It seems to me that in many cases it is better policy *not* to build a continuous floor under the hollow concrete type of dam, but merely sufficient footings under the buttresses to keep the pressure on the foundation within safe limits, paying more attention to the cut-off wall. That is, make it extremely difficult for water to get under the dam but give free exit to the tail water for all that does get through, so that no serious upward pressure can develop. This was evidently the policy in the design for the Rock Run Storage dam, Coatesville, Pennsylvania. (See Engineering and Contracting, Feb. 11, 1914, page 189).

Referring to the editorial discussion in Engineering and Contracting, April 1, 1914, page 376, I agree that the transmission of pressure through rock, concrete, and soils of varying degrees of density is a good field for experimentation. Extensive tests in that line should help remove the matter of upward pressure from the realm of guesswork. However, there has been something done along this line. In the Journal of the Western Society of Engineers for 1897, pages 449 to 508, inclusive, is a paper on this subject giving the results of tests made for a thesis at the University of Wisconsin by A. E. Broenniman and H. H. Ross. They experimented with several kinds of stone, several mixtures of cement mortar, and the stone coated with cement mortar. As I understand the results, the pressure, starting with 62.5 ins. of mercury (equal to about 70 ft. head of water) was transmitted with diminishing intensity into the stone or mortar. The pressure indicated by the curves at the farthest points where it was measured was generally equivalent to from 6 to 10 ft. head of water. The curves were extended to zero pressure a short distance beyond, at what I understand to be near the face of the stone or mortar exposed to atmospheric pressure, i. e., farthest from the face to which the maximum pressure was applied. The penetration shown to this point of zero pressure was from 13 ins. to 21 ins.

It would be interesting to know whether the pressure would be appreciable at a distance from the pressure face nearly equal to the thickness of gravity dams. The conclusions drawn by the authors of the paper were that in a dam the pressure would extend clear through, reaching zero at the down stream face, i. e., that the upper surface of water in the voids of the masonry, or at least the surface of zero pressure, curves downward from the head water surface at up stream face to the tailwater surface at the down stream face. The direction taken by the curve would, of course, depend on the porosity of the masonry.

This seems in line with the conditions as often found in earth fill dams where tests have shown in some cases that the upper surface of saturation slopes downward towards the down stream face and is, I think, generally convex upward. From the tests above referred to, I conclude that in porous concrete or stone of a uniform texture the curve would be *concave* upward. That is, that the pressure decreases more rapidly near the up stream face than it does farther in the structure. In case of a rock seam beneath the dam, it would evidently be possible to get an upward pressure much greater than zero beneath the toe of the dam if such seam admitted water freely from the reservoir but tightened up too much to permit of its free exit below the dam and close to the toe.

While making a survey of the Shawneetown, Illinois, levee, for the Internal Improvement Commission of Illinois in March, 1907, the writer had opportunity to observe some of the results of leakage under an earth dam. This levee is nearly four miles long, enclosing three sides of the town, and for most of this distance the water surface on the outside was from 7 to 15 ft. higher than the ground at the toe of the inner slope for part of the time this survey was in progress. The water had been several feet higher in January of that same year and has been still higher in previous and in subsequent years, notably in 1913 when the lower levee was voluntarily cut and the town flooded gradually to prevent a recurrence of what happened in 1898, when more than 300 ft. of the levee was carried out (or *in*) and many houses washed away and people drowned.

The top soil is largely clay, but several feet down contained more sand and during every high water stage many "seeps" or "high water springs" opened up near the toe of the inner slope. These were no doubt often due to removal of part of the top clay either on outside or inside of the levee, or both, making it easier for water to gain access to the sandy stratum on the outside and to burst up through the impervious cover on the inside. In general these seeps were controlled by piling on dirt sacks, first surrounding the seep completely and then covering it tightly with one or two or even more layers of sacks. This even proved effective in some cases where the head from outside water surface was as much as 18 ft. In free water the pressur due to 18 ft. head equals 1,123 lbs. per square foot, so it is evident there was a great reduction in pressure in passing through 60 to 100 ft. of ground, when two or three layers of dirt bags would entirely stop the flow.

In 1898 little attention was being paid to these seeps and one of them, occurring where the soil was probably more pervious than elsewhere, increased in size until so much sand and soil had been carried through that an under channel was formed, quickly increasing in size and resulting in the break above referred to and a large pocket gouged out 30 ft. below the natural surface.

Suppose a heavy type masonry dam were built on such a soil, a dam thick enough to stand the upward pressure for full head, would it prevent such an underflow, without an effective cut-off wall? Some one may say that it would compress the soil and render it less pervious. But a triangular concrete dam 20 ft. high with the center of pressure within the middle third would never load the soil with more than 3,000 lbs. per square foot vertical component, *neglecting* upward pressure of water, which, when existing, would reduce it still lower. It does not take a very good quality of earth foundation to stand 3,000 lbs. per square foot without much settlement, so, if there were a stratum of sand several feet below the base of such a heavy dam and the water in the reservoir gained access to it and also gained exit below the dam, it would probably not take any longer to undermine the structure than if it were an earth fill or a hollow concrete dam. The result would be, no doubt, a worse wreck than occurred at the Pittsfield dam "wash under."

Reverting again to the editorial comment above referred to, possibly the quite extensive investigations that have been carried on by the U. S. G. S. (and perhaps others) on the flow of underground waters might give some useful or at least suggestive information on the transmission of pressures in pervious strata

The discussion relative to varying and uncertain coefficients of friction under dams and in soil and between layers of rock are pertinent, and those are certainly subjects worthy of experimentation.

It is well to remember, however, with regard to experiments dealing with soils and rocks that these materials are exceedingly variable in kind and condition and that where the strata are not very uniform in position and texture it is easy to miss some flaw or dangerous feature because the borings and test pits explore a very small portion of the foundation material. So that however extensive our experiments with known materials under controlled conditions, we generally have some *unknowns* to deal with in foundation work. That is what makes it so interesting and gives the engineer of experience and good judgment in such lines an advantage. As has been proven more than once it will also sometimes pay to employ a geologist to pass judgment on a dam site.

With regard to sketch Fig. I, page 376, April 1, 1914, ENGINEERING AND CONTRACTING, the projection *B* under the toe of the dam no doubt would be of benefit in some cases but in other cases it might be a detriment. If the upward pressure under the dam would be, under ordinary conditions, greatest at the heel and practically zero at the toe, such a downward projection would surely increase the pressure just back of it by making the exit of the underflow more difficult. Had this scheme been used for the Austin, Texas, dam it would have had to extend to considerable depth to have been effective, as the tail water from the power house, flowing along the toe of the dam to reach the low water river channel, had cut away the rock in front of and back under the toe for several feet depth below the bottom of said toe. (See Engineering News, May 10, 1900, page 308).

I think a better way to increase the resistance to sliding when the dam is on rock is to roughen the rock surface under the entire base of the dam, full width. I presume this is generally done, but even this is not sufficient for such cases as the two Austin dams where probably the movement that started the failure in each case was at a seam or surface between different layers of rock below the base of the dam.

Much of the editorial discussion referred to and many of the dam failures seem to me to strengthen the arguments for dams with inclined up-stream faces that make use of the water pressure to *hold the dam down* instead of presenting a vertical up-stream face and so *inviting* the water to push it forward or overturn it.

The arch dam, where the conditions are suitable, is undoubtedly one of the best and most economical. I have seen the statement in print several times and have never seen it disputed that no arch dam ever failed. I think it will be found that the multiple arch dam like, or similar to the Eastwood dam, built recently in southern California at Big Bear Valley, can be safely and economically applied in many places where either the gravity or the Ambursen type would also be feasible. For five or six years I have been discussing this type of dam before my classes as a variation of the Ambursen type. I was somewhat surprised to find recently that a multiple arch dam has been in existence for over a hundred years in India. The axes of the arches are vertical. It is described in "The Practical Design of Irrigation Works," by W. G. Bligh, second edition, published in 1910 by Van Nostrand. In this book are also described several other multiple arch dams and a design by the author.

Insufficient spillway capacity has been the cause of many failures, especially of earth dams, the most notorious example of which the writer knows being the Conemaugh dam, the failure of which caused the Johnstown flood in 1889. Since the maximum flood discharge is never absolutely known and there is always a chance of its value being raised in future, why is not the spillway a good place to apply a substantial factor of safety, the size of this factor depending on the length of time for which reliable discharge data is available?

H. P. BOARDMAN,
Professor of Civil Engineering,
University of Nevada.
Reno, Nev., May 19, 1914.

Three Neglected Factors in the Designing of Masonry Dams.

Contributed by William Osmond, 220 Cumberland St., Brooklyn, N. Y.

It is not the intention of the writer to enter into a lengthy and tedious discussion of how to design safe masonry dams. Such an article would require being bound between two hard covers at least an inch apart. Following each dam failure the technical press blossoms out with articles dealing with dam design. Two weeks later the press is back to normal, describing projects of diverse kinds, methods of paving, water works and hydro-electric plants, bridges, dredges, etc. The design of masonry dams is apparently forgotten until the next large collapse. It is my desire to stir up expert discussion on the subject during this period of peace.

Influence of Overflow.—Flood conditions are the most trying to which a dam is subjected. The structure may be safe from overturning when full, but suppose many vertical feet of water are flowing over its crest; 10 ft. of overflow may reduce the factor of safety by many per cent. The depth of water to produce failure equals the height of the dam in feet multiplied by 1.33, i e., when the overflow is one-third the full head, the dam is no longer safe. The overturning moment for failure conditions is

$$M = \frac{62.5}{6} (h^3 - 3hd^2 + 2d)$$

where h is the depth of water to produce failure, and d is the depth above dam crest. The final deduction is that the overturning moment is $\frac{62.5\ h^3}{3}$.

These are simple formulas, but their very simplicity seems to have caused their logical development and the conditions they represent to have been widely overlooked. Not that they are new; there are engineers who are very familiar with their importance, but in many quarters the results prove their neglect.

Ice Thrust.—The next factor to consider in logical progression is the effect of up-stream ice. Usually this subject is disposed of by saying that ice-thrust is indeterminate; but a thing or a condition indeterminate is by no means a thing or condition to be neglected. Happy is the designer of a dam in the tropics, in the elimination of at least one menace of unknown proportion.

In allowing for ice-thrust, several factors must be considered, and they are far from simple. The strength of the current is of great importance. Ice exerts itself upon obstructions by thrusting and by impact. In a swiftly flowing river, impact is of course the controlling factor. The stress occurs by an intermittent series of blows, widely varying in force. A slow current may freeze over, which results in a complex thrust, due to current action and to the expansion-thrust due to freezing. The latter is a constant push. Should the ice break, separate, and freezing set in again, absolutely unknown and indeterminate stresses are present. The latter condition suggests that even in absolutely quiet pools ice-thrust must be taken into account. The happy solution of the ice-thrust factor is empirical. Sound judgment, based upon approximate calculations, is requisite.

Bearing on Foundation.—Need the importance of this point of the design be emphasized? Apparently, yes. Seldom does a dam break in two laterally. Almost invariably the trouble is transmitted to the base. It is not enough to know that the masonry rests upon rock. It must be ascertained by deep borings that the rock is not too thinly stratified. Often a high factor of safety is sought in the foundation. This is of no avail. Suppose the factor of safety of the dam itself is 2; of what use is a foundation factor of safety of 50? Such calculations and aims cease to be engineering. Although the rock will sustain the completed dam, failure seldom occurs through direct compression.

The pressure at the toe (downstream edge) increases from the condition of reservoir empty to reservoir overflowed. At overflow-head the stress there is infinite. At failure-level, failure occurs irrespective of the bearing power of the foundation rock.

As regards uplift, the only safe analysis is to consider it due to full head acting constantly from heel to toe. Too often it is accepted as decreasing from maximum at the heel to zero at the toe. It is probably safe to assume the latter when dealing with earth dams; but practice has shown otherwise in the case of concrete. Considering the problem in the light of simple gravity is all right in the classroom, but not where money, life, and professional reputation are at stake.

Drainage and Irrigation

Commercial Development of the Minidoka Project Power Plant, U. S. Reclamation Service.

To furnish power for pumping water to irrigate some 48,000 acres of bench lands in the Minidoka project, the Reclamation Service built a power plant at Minidoka Dam, on Snake River, Idaho. With a fall of 46 ft., this develops approximately 10,000 h.p., using water which must be allowed to flow down the river for prior rights.

Over 90 per cent of this power is required for pumping during the height of the irrigation season, but during the winter the plant would be idle except for the extensive use of power in the towns and on the farms of the project. Here electricity is used for heating and cooking, as well as for lighting and power. Information on the development of this commercial service is given by Mr. Barry Dibble, engineer, in the "Reclamation Record" for April, 1914:

The Minidoka Dam is 6½ miles south of Minidoka, Idaho, the nearest railroad point. The power house is at the north end of the dam, on the diversion channel, which is used for a tail-race. Water for running the plant is taken through 10-ft. penstocks directly from the reservoir, known as Lake Walcott, created by the dam.

The building is a two-story reinforced concrete structure, 50x150 ft., resting on piers in the diversion channel. On the first floor are five 2,000 h.p. vertical turbines connected to the penstocks. Directly above each turbine, on the second floor, is a 1,200 k.w. electric generator. On this floor also are the exciters and governors that control the turbines. The generators furnish current at 2,200 volts, which is increased to 33,000 volts by air-cooled transformers located on a gallery overhanging the second floor. This gallery also contains the switchboard for controlling the entire power house.

From the power house transmission lines carry the energy over the project. The three principal pumping plants for supplying the south side unit are from 11 to 13 miles from the power house. Some comparatively small irrigation pumps have also been installed for high-land areas on the north side of the river. A number of drainage pumps have been in use for several years to keep down the ground water until deep gravity drains could be completed.

Four substations supply power for use in the towns of Rupert, Heyburn and Burley and for groups of farmers. These towns are 13, 20 and 22 miles, respectively, from the power house. The transmission system consists of 62 miles of 33,000-volt lines and over 20 miles of 2,200-volt lines

The lands under the South Side Unit are covered by three canals, at 30, 60 and 90-ft levels. The water is lifted from stage to stage by the largest pumping stations which have ever been used for purposes of irrigation. During the irrigation season of 1913 about 120,000 acre-feet were pumped, of which 17 per cent was used on the 30-ft level, 36 per cent was used on the 60-ft level, and 17 per cent was pumped to the 90-ft. level.

The fact that this irrigation pumping uses nearly the entire capacity of the power house in the summer and leaves it idle in the winter, left an opening for a very interesting phase of development, of which advantage has been taken in supplying power to the three principal towns of the project and to the farmers and various industries. The probable demand would not have used more than a small portion of the power available for many years. The problem was to market the surplus during the winter. Electrical cooking and heating were encouraged by giving low rates, and the demand has grown rapidly. On Jan. 1, 1913, the three towns of the project were served a maximum of nearly 1,500 kilowatts, an increase of nearly 100 per cent over the amount used at the beginning of the preceding year. On Jan. 1, 1914, this had practically doubled again, being close to 3,000 kilowatts.

This use of power was so novel that at first suitable electric heaters could not be obtained, and the Government engineers were called upon to design and build them. Now heaters are shipped to the project in large quantities by several electrical firms. For the ordinary household use of electricity for lights, ironing, washing-machines, vacuum cleaners, and cook stoves, the energy costs from $3 to $4 per month, while to heat the same house it costs from $4 to $8 or $9 a month in the winter.

The revenues from the sale of electric energy have increased very rapidly, amounting in 1910 to something over $500; in 1911 to over $5,000, and in 1913 to over $15,500.

The increase in the winter demand is well illustrated by the accompanying diagram, Fig. 1, which shows the variation in the power furnished by the plant since it began operation. The peaks or high points on the curve, marking the time of greatest power demand, recur each irrigation season when almost the full capacity of the plant is utilized in pumping for irrigation. The low points on the curve represent the winter power demand and show its rapid increase.

The upper dotted line shows the tendency of the maximum summer demand to become constant, approaching the capacity of the plant. The lower dotted line indicates a tendency of the minimum winter demand to increase more and more rapidly from year to year. If these two dotted curves are extended according to these tendencies, they will meet at some point on the diagram, representing a time before the year 1920. That is, if the minimum and maximum power demands continue to increase in the same way that they have to date, by 1920 the plant would have a steady demand for its entire capacity, which would greatly increase its earning power. While such a favorable outcome is not to be expected, the diagram shows clearly the rapid increase in the demand for power since the plant was built.

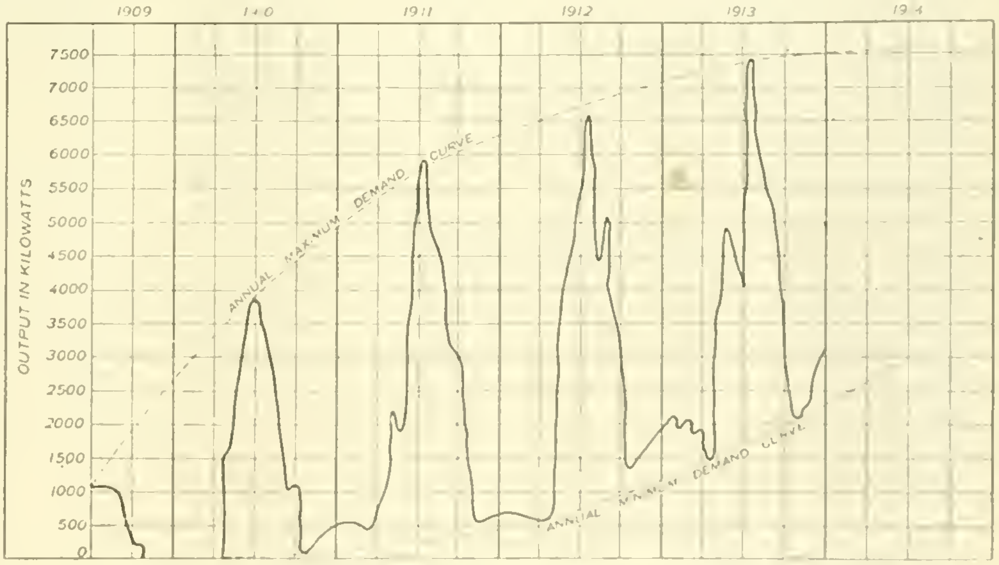


Fig. 1. Power Furnished by Minidoka Power Plant Since Construction.

The Amalgamated Sugar Co. uses over 200 kilowatts for motors in its factory at Burley. The alfalfa meal mill of the Portland Feeder Co., a new industry on the project, uses a 50 h.p. motor. Here pulp and molasses from the sugar factory are mixed with alfalfa meal and ground grain to feed over 2,000 cattle. At Heyburn a flour mill and brick yard are electrically operated.

At Rupert a handsome three-story high school with accommodations for 600 students has just been completed. This building has no chimney. Its electric heating plant has a capacity of 100 kilowatts. A blower delivers enough warmed air to fill the building every fifteen minutes. The domestic science room is equipped with individual electric hot-plates. The chemical laboratory has electric evaporating pans, water stills and other appliances. The manual training shop apparatus is run by the motor which drives the main ventilating fan. Hot water for the gymnasium baths, domestic science department, chemical laboratories and other features is provided by an electric water heater. The building is brilliantly lighted throughout.

Both Rupert and Burley have modern water systems, drawing water from deep wells by electrically operated pumps.

The power system has been operated since 1909, when power was used for pumping only. In the fall of 1910 delivery for commercial purposes began. Three eight hour shifts work at the power house and principal pumping stations. Operators are paid $75 to $85 per month, depending upon experience and responsibility. The power house is in charge of a foreman and an assistant. The three South Side pumping stations are in charge of one foreman, who also has an assistant. The transmission lines and 150-mile telephone system are patrolled and kept in repair by a foreman and lineman, who also look after considerable construction work. The smaller stations are usually taken care of by a single man at each who devotes only a part of his time to this work

General Design of Drainage Pumping Plant, South Quincy Drainage and Levee District, Illinois.

(Staff Article.)

The pumping plant illustrated is part of the drainage works, the general character and the volume of which were stated in our Current News Section for May 6, 1914. A main ditch bisecting the area collects the flow of the interior ditch and drain system and carries it to the levee beyond which the water is then carried by sluicing or pumping. The illustrations show the sluice way and pumping plant structures. They are located behind the heel and under the line and consist of a suction bay, sluice way and appurtenances, discharge bay, pump foundations and pumps and a pump house. The several structural features are described fully by the drawings.

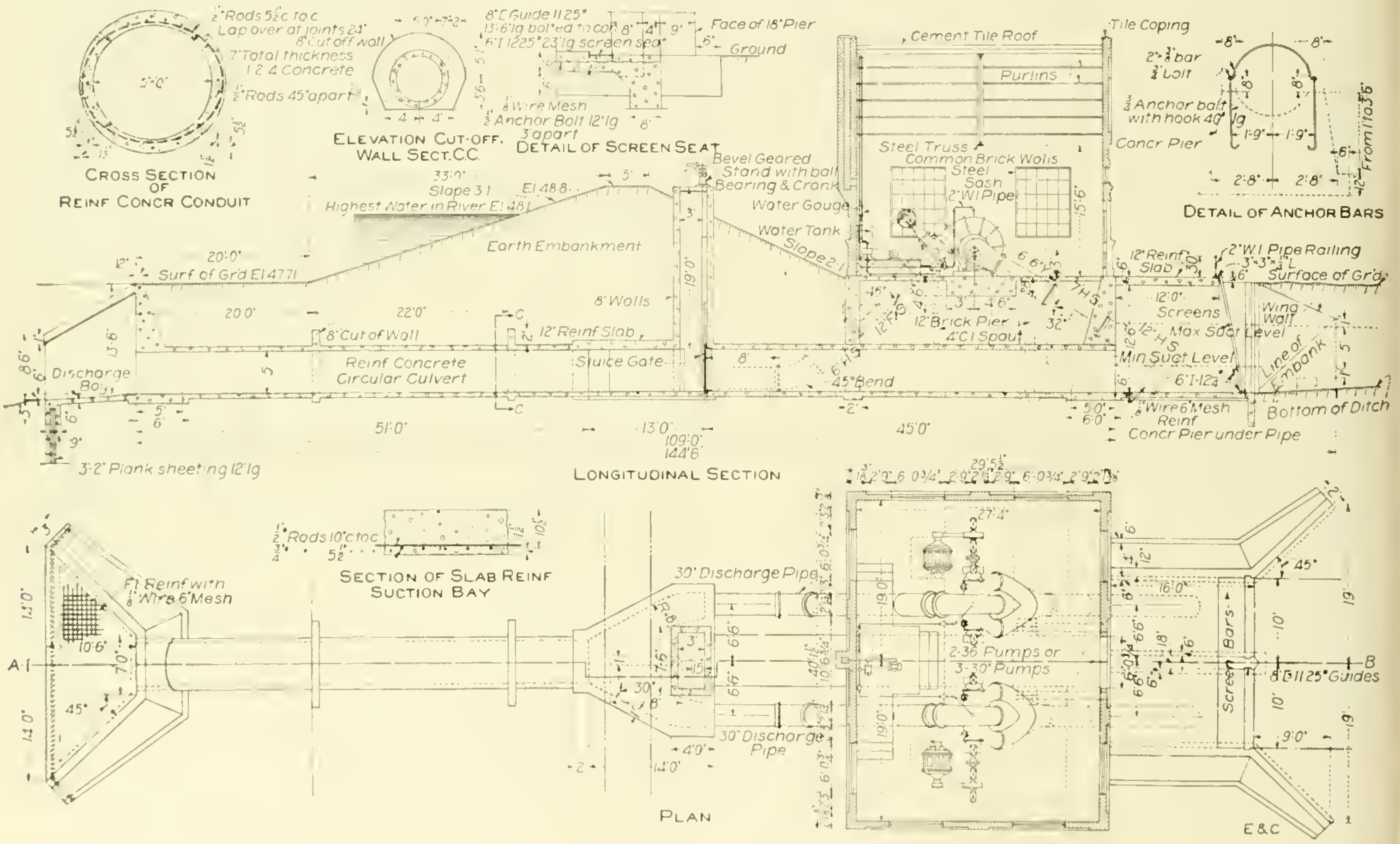


Fig. 1. General Plan and Details of Drainage Pumping Plant for the South Quincy Drainage and Levee District, Illinois.

In constructing the substructure it is required that no levee be built at this point until the concrete work is completed. It is also required that all trenches and pits for the concrete structures be excavated with vertical sides held by sheeting and to dimensions no larger than are necessary to accommodate the structure and the sheeting. The culvert trench is required to have its bottom rounded to the culvert arc so that no outside forms are needed to shape the concrete; if this soil be too unstable for such shaping a footing or base for the culvert is to be provided. Exactness in excavation is particularly required. The plans call for about 15,000 cu. yds. of excavation.

Besides care not to disturb the natural soil unnecessarily, great care is demanded in the backfilling. All backfilling is to be with shovels and the earth is to be tamped from the bottom up using not fewer than two tampers to each shoveler. Around pipes and culverts selected material for given stress is required for backfilling and particularly careful tamping is demanded. Attention is directed to the toe and heel walls and to the cut-off walls or collars provided to prevent percolation along the outside walls of the culvert.

For sluice concrete a 1½-in. stone or gravel 1:3:6 mixture is to be used and for reinforced concrete a ¾-in. stone or gravel 1:2:4 mixture. In other details the concrete materials and manufacture are those required by good practice in concrete building work. The characteristics of the reinforcing required are indicated by the drawings. Steel of 60,000 to 70,000 lbs. ultimate strength is required and precision of spacing is particularly demanded. Another requirement is the use of long bars to minimize the number of laps; these are to be staggered and kept as far apart as is practicable and are also to be at least 30 diameters overlap. No welded bars are permitted. Forms are required to be such as to give not only true shapes and dimensions, but also a smooth surface. The inside of the culvert in particular is required to be finished smooth and then given a wash coat of neat cement.

The general arrangement of the pumps and piping and of the gates and other appurtenances is shown by the drawing. The requirements for pipe laying are:

> Pipes shall be laid in trenches carefully prepared for them and shall be securely supported in position until all joints have been poured and the earth has been thoroughly compacted under and around them. Where pipes pass through the foundation walls, openings shall be left so that no weight will be brought to bear upon the pipe due to any settlement in the masonry structures. Suction pipes must be absolutely air-tight; discharge pipes must be water-tight. The engineer will test for these requirements before accepting the work, and the contractor will be required to make good any defects which may be found.
>
> The best quality of jute packing shall be used for calking pipe joints, and shall be well driven up so as to leave, in all cases, a space for lead of 2¼ ins. in length in hubs and 3 ins. at each end of sleeves.
>
> Before calking, all parts of the joints shall be thoroughly cleaned with a bellows and brush, and no lead shall be run in unless the joint is clean. All joints must be calked in the trench, and no calking on the surface of the ground will be allowed.
>
> The spigots of the pipes shall be carefully adjusted in the hubs so as to give a uniform space all around for the lead joints.
>
> The best soft lead in pigs shall be used for the joints, and when melted shall be kept free from dross and all impurities, and each joint shall be run at one pouring. No plugging or repairing of a joint shall be done with cold lead or otherwise after the joint is first run. The lead shall be driven up and condensed with proper tools, and in any case in which a joint is not done to the satisfaction of the engineer the contractor must remove the lead and packing and repack and relead the joint.
>
> All pipes and castings shall be carefully swept and cleaned as they are laid and shall be freed from any earth, stones or rubbish which may find place in them before or during the operation of laying, and every open end shall be closed before leaving the work at any time.

The pumping machinery proper consists of two double suction centrifugal pumps of 60,000 gals. per minute capacity operated by two 250 hp. motors, 440 volts, 3-phase, 550 revolutions per minute.

The engineers for this plant are the Edmund T. Perkins Engineering Co., Chicago, Ill.

Dipper Dredge for Panama Canal.—The "Paraiso," a 15 cu. yd. dipper dredge, twin of the "Gamboa," now at work in Culebra cut, has sailed from New York in tow of a tug. These dredges were constructed by the Staten Island Shipbuilding Co. The dippers are 10 ft. 9½ ins. in height, from the bottom band to the upper edge of the lip, and each has a capacity of 15 cu. yds. They are built with manganese steel lips and dipper-door hinges of forged steel, 6x10 ins. in cross section. They are hung on a 3¼ in. single-part, wire rope hoisting cable. The handle is 72 ft. long with top and bottom bars measuring 2x12 ins. in cross section, and weighs 40½ tons. The dipper boom is 62 ft. long, weighs 66½ tons and is of plate-girder construction. The dredges can dig to a depth of 50 ft. below the surface of the water.

Earth Slides on the Calumet-Sag Branch of the Chicago Main Drainage Channel.

The considerable development of earth slides on the Calumet-Sag Channel of the Sanitary District of Chicago is discussed in a recent special committee report to the Trustees of the District. The engineer members of this committee were E. J. Killey, Assistant Chief Engineer; and Philip Harrington, Resident Engineer. The following are the sections of their report which relate specifically to the Calumet-Sag slides:

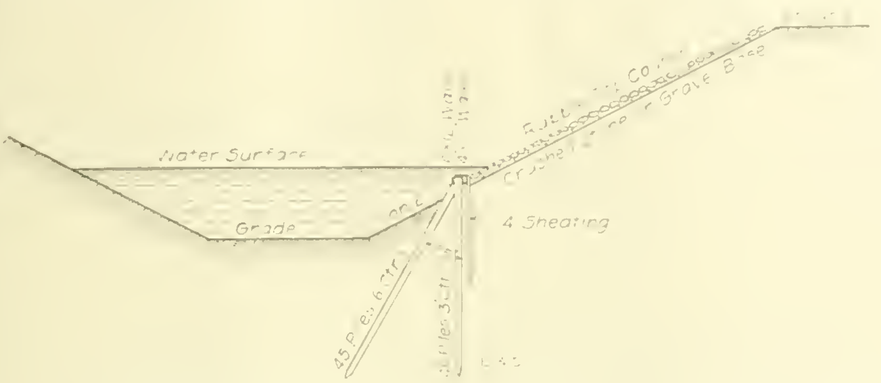


Fig. 1. Method of Slope Paving, North Shore Channel, Chicago Drainage Canal.

The usual formation encountered in the Sag valley, where the Calumet-Sag Channel is now being constructed, is a shallow layer, not exceeding 5 ft., of a clay having a light yellowish or buff tint, just below the top soil, below which is found the blue clay which forms the body of the drift. This blue clay sometimes becomes very dense and is excavated with difficulty. Below this is often encountered a formation of a hard brown clay, sometimes containing gravel and small boulders partially cemented. This material is extremely difficult to excavate, and often necessitates blasting. Occasionally a water-bearing layer of blue clay containing a large percentage of sand and silt and varying in thickness from 1 to 5 ft. is encountered. This silty clay (locally termed "bull-liver") is very unstable and when exposed has a tendency to flow. Its supporting power is of course very small, and this formation has constituted one source of trouble in maintaining the canal banks. Another source has been seams of clay resembling shale. This clay tends to disintegrate and crumble upon exposure to the atmosphere, which leaves the bank unsupported. In the swampy regions, of which there is a large area along the route, extensive peat beds are found. This peat formation sometimes occurs at a depth of 10 to 20 ft. below the ground surface, although usually it constitutes the top soil. This peat, when relieved of the moisture which it has absorbed, is very light in weight and incapable of supporting a high bank. As a result, considerable difficulty has been encountered in designing the canal slopes and disposing of the spoil to meet this condition.

Three types of sliding ground have been encountered as follows: (1) Structural breaks resulting from inability of a layer of drift to hold the weight of the overhead bank. (2) Normal or gravity slides. (3) Surface erosion.

Structural breaks occur at points where a layer of shale upon exposure to the atmosphere disintegrates and crumbles. A crack or fissure then develops in the bank, sometimes at a distance of 200 to 300 ft. from the channel center line and this crack gradually widens and deepens as the bank moves slowly into the channel. Instead of a layer of shale, the prime cause may be a peat stratum, or it may be a soft, silty or unstable clay.

The normal or gravity slide results from the movement of the overhead bank upon a slippery layer of clay or other material the line of stratification of which is clearly defined. It is due almost entirely to an excavated slope steeper than the angle of repose of the particular formation then being excavated, and in many cases has been further aggravated by the superimposing on the berm heavy spoil banks at a comparatively small distance from the slope.

Surface erosion is a gradual sloughing of the surface of the slopes due to the weathering action of the elements. In addition to rain and wave action, the action of frost is a contributing cause of slides of this character. The freezing and thawing of the bank tends further to aggravate the sloughing of the surface of the slopes, so that the sowing of shallow rooted grasses and like vegetation is not in all cases a sufficient preventive means.

Where slides occur due to the failure of an underlying stratum of material to hold up the unbalanced pressure or where the bank created by the same operation shows movement upon an underlying stratum of a different material, the only remedy which can be advised is the flattening of the slopes until sufficient weight has been removed to establish a state of equilibrium; in other words, until the angle of repose for the material then moving has been attained. For this reason, it has been maintained that waste or spoil banks should be placed at a considerable distance from the banks in all cases where preliminary borings show indications of formations of this character and the side slopes of the channel designed flat enough to prevent movement of the bank. The distance that the spoil bank should be placed from the channel, and the angle of inclination of the side slopes are subjects of conjecture, and may be arrived at with any degree of accuracy only by one having long experience with similar work and with the particular class of material to be encountered. It is far from an exact computation. However, it may be generally said that the angle of inclination of a slope to prevent movement is in most cases steeper than the angle necessary to retard and stop the movement of a bank which has commenced to slide. For this reason it can readily be seen that it is far more economical to excavate the side slopes to the necessary inclination as the excavation is carried down to grade, than it is to allow the material to slide and excavate it afterwards. Not only does the latter method tend to increase the total yardage in any cross section, but it also involves other difficulties such as extricating machinery, track, etc., with the accompanying loss and expense.

The Calumet Sag Channel, having a cut of about 40 ft., was designed with slopes in earth of 1 vertical on 2 horizontal, with the spoil banks removed from 50 to 100 ft. from the top of the slope. In some cases this has proven an ample margin of safety, while in others slides have occurred at points where the material is soft and of unstable character. These conditions have been recognized and much of the later work that has been awarded by contract, has called for 1 vertical on 3 horizontal slope, with the spoil banks removed 400 ft. from the top of the slope. Moreover, in one case, where the material is exceptionally soft, the contractor has been ordered to place no spoil on the berms, but to transport it approximately one-half mile to a specially provided dump.

To prevent slides due to surface erosion, the remedy which has been found most successful and efficient has been to rip-rap the side slopes with large sized rubble in a layer 8 to 10 ins. thick placed upon a 12-in. layer of crushed stone or gravel. The crushed stone or gravel forms a cushion between the rubble and natural bank and at the same time provides a water course for the surface water which would otherwise flow down the slope. It also tends to prevent the frost making any great headway into the slopes, and thus obviates the results of thawing. In some cases it has been found advantageous to drive a pile and timber foundation with batter piles in front to act as a brace for this rip-rap slope pavement. This foundation serves a two-fold purpose, it acts as a support for the rip-rap, and it tends to prevent further movement of the banks due to the overhead weight. It has been tried on the North Shore Channel, Fig. 1, and, so far, has proven entirely successful. However, where rip-rap can be placed before the water has been turned into the channel, it is better and more economical to carry the slope paving to a solid foundation, such as rock or very hard earth. This latter method is now being employed on practically the entire length of Calumet-Sag Channel. Fig. 2, the rock necessary for the rubble being excavated from the canal.

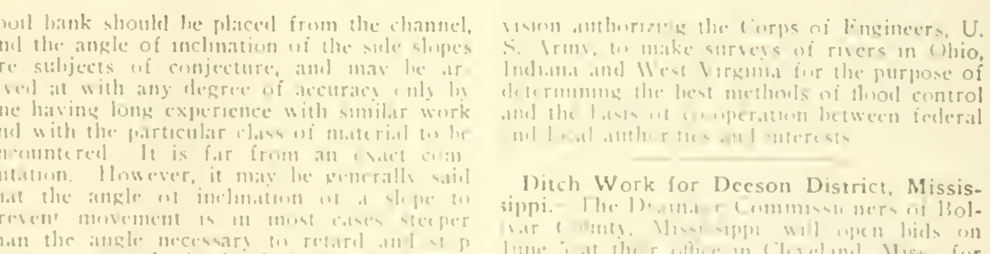


Fig. 2. Method of Slope Paving, Calumet Sag Branch, Chicago Drainage Canal.

Rivers and Harbors Bill and Surveys for Flood Control.—The subcommittee which has been considering the federal rivers and harbors appropriation bill passed recently by the house of representatives reported the measure last Thursday to the senate committee on commerce. The bill as reported carries appropriations of nearly $51,000,000, an increase of $8,000,000 over the amount in the house act. One feature of the bill is the provision authorizing the Corps of Engineers, U. S. Army, to make surveys of rivers in Ohio, Indiana and West Virginia for the purpose of determining the best methods of flood control and the basis of cooperation between federal and local authorities and interests.

Ditch Work for Deeson District, Mississippi.—The Drainage Commissioners of Bolivar County, Mississippi will open bids on June 5 at their office in Cleveland, Miss., for the excavation of the following canals in the Deeson Drainage District:

Name of ditch	Length, miles	Av. cut, ft.	Bottom width, ft.	Excavation, cu. yds.
Main Ditch	[illegible]	[illegible]	10	241,630
Lateral A	[illegible]	[illegible]	10	16,000
Lateral [illegible]	[illegible]	[illegible]	6	10,000
Lateral E	[illegible]	[illegible]	6	23,000

The main ditch and Lateral A follow a natural channel and will have to be cleared of cypress and other timber for about three-fourths of its distance. Laterals E and F will not follow a natural channel and the clearing will be light. W. W. Boone, Cleveland, is Engineer.

Railways

A Rating Table for Excavation With Pick and Shovel.

Contributed by L. K. Sherman, N. Am. Soc., C. E., Chicago, Ill.

The accompanying diagram and tables represent the amount of excavation of various materials which will be performed in a ten-hour day by the average laborer working under good supervision. In making this compilation the writer has compared a large number of data from many sources with figures obtained in his own experience on construction. As might be expected there is wide divergence in such published data.

The curves in the diagram based on a rational relation of one class of material to another as regards the amount of work or power required in picking or shovel cutting and the power required in casting up materials of different weights. The output of excavation is proportional to the amount of power or work required to move a cubic yard of the material. Let the amount of work or power to cut into and fill the shovel with sand be called unity. Then for other materials the relative power to cut out and place on the shovel will from experience be as in Table 1.

TABLE I.—POWER TO PICK, LOOSEN AND CUT ONTO SHOVEL.

Sand	P=1.0
Gravel, loose	P=1.5
Earth, medium	P=2.0
Clay, light	P=3.0
Clay, dry, hard	P=4.5
Clay, wet, heavy	P=5.0
Hard pan	P=6.0

The work or power to lift or cast up the material after the shovel is filled is proportional to the weight of material and height cost or which is the same, the depth of cut. Then if W is the weight, the relative power to cast up material to different heights H will be as follows:

Sand	W H where W=1.0
Gravel	W H where W=1.9
Earth, medium	W H where W=0.8
Clay, light	W H where W=1.1
Clay, dry	W H where W=1.1
Clay, wet	W H where W=1.3
Hard pan	W H where W=1.12

The total power to shovel and cast any material is $P+WH$. The output is inversely proportional to the power or work required. The output of any material by hand excavation in cubic yards per man per 10 hours is

$$\text{Cubic yards} = \frac{30}{P + .3WH}$$

The constants 30 and .3 are empirical and like the relative values of P have been selected to correspond with the best data available on excavation of various materials at different depths of cut.

The curves in the diagram Fig. 1 are platted according to the above formula with coefficients P and W as previously noted. The letters represent observations from various published statements and are not equally reliable or comparable. The curves do not attempt to average the data but correspond with the writer's experience and some of the most definite of the published data. Table II shows the number of cubic yards an average laborer should excavate and cast out, at various depths in ten hours while working at the depths stated. Table III shows the average number of cubic yards per 10-hour day that an average laborer should excavate working from the surface to the depth stated. This figure for the same material is naturally somewhat greater than given in Table II. These figures may be increased by 30 per cent for rapid workers and may be decreased 30 per cent for inefficient workmen. The foregoing material may be now definitely classified as follows:

Sand.—Weight, 3,000 lbs per cubic yard slightly damp. In natural bed. Not over 15 per cent clay.

Gravel.—Weight, 3,000 lbs. per cubic yard. Loose, as excavated material.

TABLE II.—CUBIC YARDS PER MAN PER 10 HOURS AT STATED DEPTHS.

	0 ft. to 3 ft.	3 ft. to 5 ft.	5 ft. to 8 ft.	8 ft. to 10 ft.	10 ft. to 15 ft.
Sand	21.2	14.5	10.7	8.5	5.2
Gravel, loose	15.4	11.8	9.2	7.7	4.9
Earth	12.8	10.5	9.0	7.5	4.9
Light clay	8.9	7.3	6.0	5.2	3.8
Dry clay	6.4	5.3	4.7	4.1	3.2
Wet clay	5.4	4.7	4.2	3.5	2.7
Hard pan	4.6	4.2	3.7	3.3	2.7

Earth.—Weight, 2,400 lbs. per cubic yard. Slightly damp, in natural bed, easily plowed, little or no pick work required. Would require some sheeting in trenches over 6 ft. deep.

Clay (light).—Weight, 3,300 lbs. per cubic yard. Slightly damp, easily plowed. Not stiff or very cohesive, corresponds to yellow clay lying below the black soil and above the blue clay in vicinity of Chicago. Would require some sheeting in trenches over 6 ft. deep. Little pick work required.

Clay (dry, hard).—Weight, 3,300 lbs. per cubic yard. Requires pick work equal to one-third time spent in shoveling and casting. No sheeting required at any depth. Corresponds to material on top of ravines along the lake shore in Lake County, Ill. Hard plowing. Adobe in this class.

TABLE III.—AVERAGE EXCAVATION IN CUBIC YARDS PER 10 HOURS FOR CUTS FROM SURFACE TO STATED DEPTHS.

	0 ft. to 3 ft.	0 ft. to 5 ft.	0 ft. to 8 ft.	0 ft. to 10 ft.	0 ft. to 15 ft.
Sand	21.2	18.1	15.1	13.6	10.7
Gravel, loose	15.4	13.7	11.8	10.8	8.8
Earth	12.8	11.7	10.5	9.7	8.1
Light clay	8.9	8.1	7.3	6.7	5.8
Dry clay	6.4	5.9	5.4	5.1	4.5
Wet clay	5.4	5.1	4.7	4.4	3.8
Hard pan	4.6	4.4	4.2	3.9	3.5

Clay (wet).—Weight, 3,900 lbs. per cubic yard. Tough and cohesive, has to be cut out in pieces. Slightly sticky, would require substantial sheeting. Corresponds to the underlying "blue clay" of Chicago. Gumbo in this class.

Hard Pan.—Weight, 3,360 lbs. per cubic yard. Requires picking equal to one-half the time spent in shoveling and casting.

The use of the relative coefficient P is suggested as a simple and definite means of describing or designating any class of earth excavation.

The jog in the curves (Fig. 1) at depth of 9 ft. represents an allowance of $P=1$ on account of extra labor of shovel cutting done to recasting from a platform. As a matter of fact no recasting may be done at the 9 ft. depth or even 14 ft. depth but the output per man will not be increased over the quantity shown by the diagram.

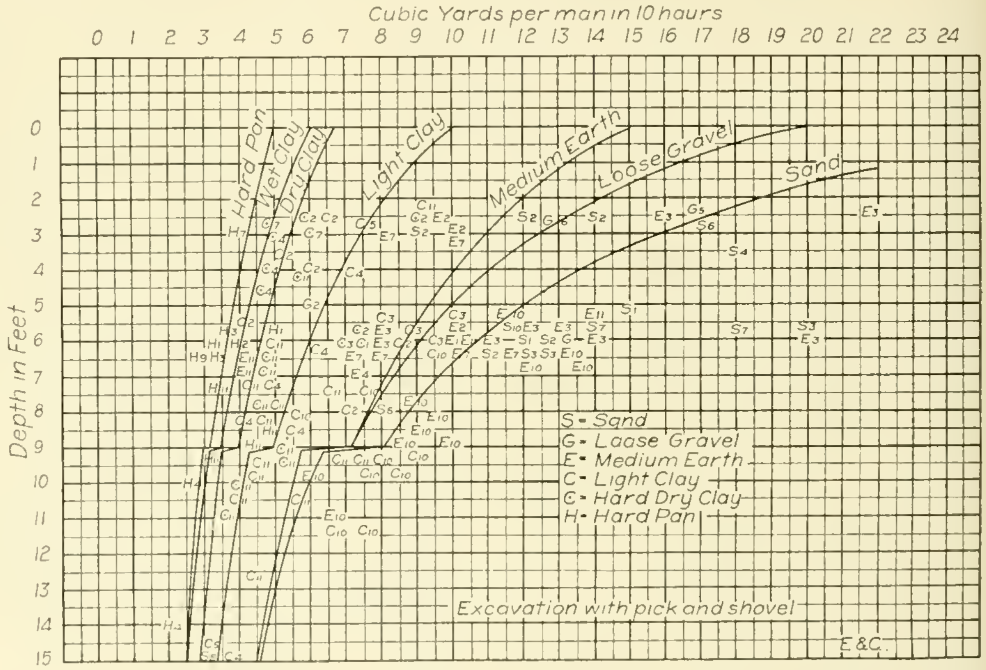


Fig. 1. Excavation With Pick and Shovel.

The recorded data platted on Fig. 1 are designated by a letter for the class of material. The number following the letter refers to the source from which the data were obtained, as follows: (1) American Engineers' Pocket Book; (2) Cost Data, Gillette; (3) Earth Work Cost, Gillette; (4) L. K. Sherman; (5) Windette, Journal West. Soc. Engrs.; (6) Concrete Costs, Taylor & Thompson; (7) Orrock; (8) Prelini; (9) Engineering and Contracting (December, 1908), Atlantic, Iowa, Sewers, M. A. Hall; Centerville, Ia., Sewers, M. A. Hall; (10) Engineering and Contracting; (11) Engineering and Contracting.

Laying Brick: Rate of Performance.—The following table compiled from information given in the Canal Record gives the performance of a brick laying gang working on the brick floor of the quay at the Balboa terminals, Panama:

Item.	
No. of brick laid by 5 men in 10 hours	96,000
No. of brick laid per man per hour	1,920
No. of men handling brick, mixing and carrying mortar, etc.	62
Cost of laying, per 1,000 brick, at 13c per hour, for layers	7c

The laying squad consisted of five West Indian masons, serving in relays, three men working at the same time with two resting in reserve. The paving was laid between the inner rail of the track along the face of the quay and the inner coping and covered an area of 2,222 sq. yds., the dimensions being approximately 50 ft. by 400 ft.

TABLES FOR ESTIMATING GRADING QUANTITIES ON REVALUATION

By M. H. Brinkley, C. E.

Assistant Engineer, Railroad Commission of California.

EXPLANATION OF TABLES

On account of the large amount of railway valuation which is to take place in the near future, quick methods of estimating grading quantities will be necessary. This is especially true if cuts and fills are measured up in the field. On most of the railways of the country adequate records of original cross-section quantities and cross-slopes or often center ground-line profiles do not exist. In such cases, the usual cross-section on revaluation is a four-point section, two points being at the intersection of the roadbed plane at the grade line with the side slopes and the other two points at the intersection of the side slopes with the ground surface.

The table shown herewith was drawn up by the writer and adopted on the revaluation of the New York Central R. R. and was also used on canal excavation in connection with the revaluation of hydraulic properties carried forward by J. G. White & Co. in California. This table is applicable to any side slopes or width of roadbed, the only condition being that the section shall be a four-point section with level base. Most sections on revaluation are of that type and irregular sections are infrequent except on very heavy work.

A section is assumed to be uniform for a volume extending halfway to the adjoining sections on either side. By looking twice in the table, the yardage per 100 ft. for any section may be taken out and placed in the notebook opposite the section. It is then a simple operation to multiply by the length, which is equal to one-half of the distance between the two adjoining sections. The yardage at each section is not exact for the volume considered, but the total yardage in any cut or fill is exact.

Take as an example the case where the field notes are as follows:

$$\frac{+7.0}{18.0} \quad \frac{0.0}{12.0} \quad \frac{0.0}{11.0} \quad \frac{+9.0}{19.0}$$

Then (see diagram on accompanying cut),

$B = 23.0$
$d_2 = 8.0$
$d_1 = 6.0$
$B + d_1 = 29.0$

Note also that the value $B + d_1$ is taken from the above field notes as follows:

$B + d_1 = 11 + 18 = 29$

The latter operation is quicker than the former and should be used. It needs only to be mental.

For $B + d_1 = 29$ and $h_2 = 9$ the corresponding volume from the table is 483 cu. yds. Similarly $B + d_2 = 12 + 19 = 31$, $h_1 = 7$; and corresponding volume, from the table, is 402 cu. yds.

The total volume per 100 ft. $= 483 + 402 = 885$ cu. yds.

If, instead of the volume extending to the halfway points on each side of a section, the bounding limits are taken at the sections, the yardage is easily found by taking one-half the sum of the two volumes. This gives the average yardage per 100 ft. and with the distance known, the total exact yardage is obtained.

On the tabulation sheet the column usually containing end area figures may be occupied by the items cubic yards per 100 ft., another column by distances and a third column by the final yardage results.

It should be noted in the tables that $B + d_1$ or $B + d_2$ and H_1 or H_2 may be read off either in the vertical column or at the top of the sheet and are interchangeable.

These tables have particular application to sections which are the result of remeasurement of earthwork quantities. On this account they are suitable for use on the revaluation of railways and canals. But the underlying principles involved are such that the tables can be made of very broad application. They are made up by taking $\frac{100}{54}$ of the product of two numbers. On this basis they can be used in estimating grading quantities from any cross-section however irregular, although particularly applicable to four point sections. For instance, by the ordinary methods, areas are computed by multiplying distances between readings by the sum of the two bounding heights. This is done across the figure and from the sum of the results is subtracted twice the areas below the slopes, obtained in the same manner. One-half of the remainder is the area of the figure. If the yardage per 100 ft. is wanted it is found by taking 100/27 of the area. This amounts to taking 100/54 of the remainder above.

By the improved method, the distances between readings and the sum of the two bounding heights are looked up in the table and the yardage read off. The results are added. In the same way the yardages below the slope lines are obtained and their sum subtracted from the result above. The remainder is the yardage per 100 ft. This gives much quicker results than by the ordinary methods.

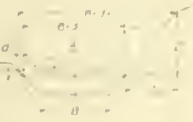
The table for railways is made up to even feet as that is as close as measurements are usually made in the field on revaluation. The uncertainties of the work are such that it is a useless refinement to take measurements closer than the nearest foot. If rod readings to tenths are taken interpolation is easy, as the intervals are entirely regular.

The table for canals was made up to the nearest half foot on account of there being a greater percentage of error in taking the nearest foot for small canals, although only in small ditches is this refinement necessary.

By the use of these tables there is a very great saving of time and labor over the old methods for four point sections, and astonishing progress can be made. But the work is also rapid for irregular sections.

TABLE OF VOLUMES FOR CANALS

This table gives cubic yards per 100 ft. of cross-section above water surface. The yardage in cross-[illegible] below water surface [illegible] is a run uniform for each base and side slope and may be calculated extra. The table also applies taking the bottom of the canal as the base.



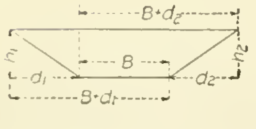
$$V = \frac{100}{54}[(B+d_1)h_2 + (B+d_2)h_1]$$

Cubic Yards per One Hundred Feet.

h_1 or h_2

$B+d_1$ or $B+d_2$	0'	1'	1'	2	2	3'	3	4'	4	5'	5'	6	6	7	7	6	8'	9	9'	10'
4	1	7	11	15	19	22	26	30	33	37	41	44	48	[illegible]	[illegible]	[illegible]	63	67	71	74
4	4	8	13	17	21	[illegible]	29	33	37	[illegible]	[illegible]	[illegible]	51	[illegible]	[illegible]	[illegible]	71	[illegible]	79	[illegible]
5	5	9	14	19	23	[illegible]	32	37	42	46	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	74	79	[illegible]	88	93
5	5	10	[illegible]	[illegible]	[illegible]	31	[illegible]	41	46	51	[illegible]	61	[illegible]	71	[illegible]	[illegible]	[illegible]	[illegible]	97	102
6'	6	11	17	[illegible]	[illegible]	33	39	44	[illegible]	[illegible]	61	[illegible]	[illegible]	78	[illegible]	[illegible]	91	[illegible]	[illegible]	111
6	6	12	18	[illegible]	30	36	42	48	[illegible]	60	66	72	[illegible]	84	90	96	[illegible]	[illegible]	114	[illegible]
7	6	13	19	26	32	39	[illegible]	[illegible]	[illegible]	[illegible]	71	[illegible]	84	91	97	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
7	7	14	21	[illegible]	35	42	48	56	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	104	111	[illegible]	[illegible]	[illegible]	[illegible]
8	7	[illegible]	22	30	[illegible]	44	[illegible]	[illegible]	[illegible]	[illegible]	81	[illegible]	[illegible]	104	111	[illegible]	[illegible]	[illegible]	141	[illegible]
8'	8	16	24	[illegible]	[illegible]	47	[illegible]	[illegible]	71	[illegible]	[illegible]	94	[illegible]	[illegible]	118	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
9'	[illegible]	17	[illegible]	[illegible]	42	50	58	67	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	117	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	167
9'	9	18	[illegible]	[illegible]	44	53	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	141	[illegible]	[illegible]	[illegible]	[illegible]
10'	9	19	[illegible]	[illegible]	[illegible]	56	[illegible]	74	[illegible]	[illegible]	[illegible]	111	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	185
10	9	19	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	78	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
11'	10	[illegible]	31	41	[illegible]	61	71	82	92	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
11	11	21	[illegible]	[illegible]	[illegible]	64	75	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	21
12	11	[illegible]	3	4	[illegible]	67	78	[illegible]	[illegible]	111	[illegible]	[illegible]	11	[illegible]	[illegible]	17	[illegible]	2	[illegible]	[illegible]
1	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	81	[illegible]	101	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
13	12	24	[illegible]	[illegible]	60	[illegible]	81	[illegible]	10	1	1	[illegible]	[illegible]	[illegible]	1	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
13	12	24	[illegible]	[illegible]	[illegible]	[illegible]	88	[illegible]	11	1	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	21	[illegible]	[illegible]	2
14	1	[illegible]	[illegible]	[illegible]	[illegible]	78	91	104	11	1	11	1	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
11	11	[illegible]	[illegible]	1	[illegible]	81	[illegible]	[illegible]	[illegible]	1	11	[illegible]	[illegible]	[illegible]	[illegible]	21	[illegible]	[illegible]	[illegible]	[illegible]
15	11	[illegible]	1	[illegible]	[illegible]	[illegible]	97	111	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
15	11	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	100	11	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
16	[illegible]	[illegible]	14	[illegible]	71	[illegible]	104	119	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
16'	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	122	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	1	[illegible]	[illegible]	[illegible]	[illegible]
17	16	[illegible]	[illegible]	[illegible]	78	[illegible]	110	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
17'	16	[illegible]	[illegible]	[illegible]	[illegible]	97	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
18	16	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	1	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
18'	17	[illegible]	51	[illegible]	[illegible]	[illegible]	120	[illegible]	[illegible]	17	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
19	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	111	[illegible]	1	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	17	[illegible]	[illegible]
19'	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	271	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
20'	[illegible]	[illegible]	[illegible]	74	[illegible]	111	[illegible]	118	[illegible]	[illegible]	[illegible]	[illegible]	211	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]

ESTIMATING GRADING QUANTITIES ON REVALUATION



$$V = \frac{100}{54}\left[(B+d_1)h_2 + (B+d_2)h_1\right]$$

(Cubic Yards per 100 Feet)

$B+d_1$ or $B+d_2$

h_1 or h_2	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	23	24	25
11	20	41	61	81	102	122	143	163	183	204	224	245	265	285	306	326	346	367	387	407	428	448	469	489	509
12	22	44	67	89	111	133	156	178	200	222	245	267	289	311	333	356	378	400	422	444	467	489	511	533	556
13	24	48	72	96	120	144	169	193	217	241	265	289	313	337	361	385	409	433	457	481	506	530	554	578	602
14	26	52	78	104	130	156	182	207	233	259	285	311	337	363	389	415	441	467	493	519	544	570	596	622	648
15	28	56	83	111	139	167	194	222	250	278	306	333	361	389	417	444	472	500	528	556	583	611	639	667	694
16	30	59	89	119	148	178	207	237	267	296	326	356	385	415	444	474	504	533	563	593	622	652	681	711	741
17	31	63	94	126	157	189	220	252	283	315	346	378	409	441	472	504	535	567	598	630	661	693	724	756	787
18	33	67	100	133	167	200	233	267	300	333	367	400	433	467	500	533	567	600	633	667	700	733	767	800	833
19	35	70	106	141	176	211	246	281	317	352	387	422	457	493	528	563	598	633	669	704	739	774	809	844	880
20	37	74	111	148	185	222	259	296	333	370	407	444	481	519	556	593	630	667	704	741	778	815	852	889	926
21	39	78	117	156	194	233	272	311	350	389	428	467	506	544	583	622	661	700	739	778	817	856	894	933	972
22	41	81	122	163	204	244	285	326	367	407	448	489	530	570	611	652	693	733	774	815	856	896	937	978	1019
23	43	85	128	170	213	256	298	341	383	426	469	511	554	596	639	681	724	767	809	852	894	937	980	1022	1065
24	44	89	133	178	222	267	311	356	400	444	489	533	578	622	667	711	756	800	844	889	933	978	1022	1067	1111
25	46	93	139	185	231	278	324	370	417	463	509	556	602	648	694	741	787	833	880	926	972	1019	1065	1111	1157
26	48	96	144	193	241	289	337	385	433	481	530	578	626	674	722	770	819	867	915	963	1011	1059	1107	1156	1201
27	50	100	150	200	250	300	350	400	450	500	550	600	650	700	750	800	850	900	950	1000	1050	1100	1150	1200	1250
28	52	104	156	207	259	311	363	415	467	519	570	622	674	726	778	830	881	933	985	1037	1089	1141	1193	1244	1296
29	54	107	161	215	269	322	376	430	483	537	591	644	698	752	806	859	913	967	1020	1074	1128	1181	1235	1289	1343
30	56	111	167	222	278	333	389	444	500	556	611	667	722	778	833	889	944	1000	1056	1111	1167	1222	1278	1333	1389
31	57	115	172	230	287	344	402	459	517	574	631	689	746	804	861	919	976	1033	1091	1148	1206	1263	1320	1378	1435
32	59	119	178	237	296	356	415	474	533	593	652	711	770	830	889	948	1007	1067	1126	1185	1244	1304	1363	1422	1482
33	61	122	183	244	306	367	428	489	550	611	672	733	794	856	917	978	1039	1100	1161	1222	1283	1344	1406	1467	1528
34	63	126	189	252	315	378	441	504	567	630	693	756	819	882	944	1007	1070	1133	1196	1259	1322	1385	1448	1511	1574
35	65	130	194	259	324	389	454	519	583	648	713	778	843	908	972	1037	1102	1167	1231	1296	1361	1426	1491	1556	1620
36	67	133	200	267	333	400	467	533	600	667	733	800	867	933	1000	1067	1133	1200	1267	1333	1400	1467	1533	1600	1667
37	69	137	206	274	343	411	480	548	617	685	754	822	891	959	1028	1096	1165	1233	1302	1370	1439	1507	1576	1644	1713
38	70	141	211	281	352	422	492	563	633	704	774	844	915	985	1055	1126	1196	1267	1337	1407	1478	1548	1618	1689	1759
39	72	144	217	289	361	433	505	578	650	722	794	867	939	1011	1083	1155	1228	1300	1372	1444	1517	1589	1661	1733	1806
40	74	148	222	296	370	444	518	593	667	741	815	889	963	1037	1111	1185	1259	1333	1407	1481	1556	1630	1704	1778	1852
41	76	152	228	304	380	456	531	607	683	759	835	911	987	1063	1139	1215	1291	1367	1443	1518	1594	1670	1746	1822	1898
42	78	156	233	311	389	467	544	622	700	778	856	933	1011	1089	1167	1244	1322	1400	1478	1556	1633	1711	1789	1867	1944
43	80	159	239	319	398	478	557	637	717	796	876	956	1035	1115	1194	1274	1354	1433	1513	1592	1672	1752	1832	1911	1991
44	81	163	244	326	407	489	570	652	733	815	896	978	1059	1141	1222	1304	1385	1467	1548	1629	1711	1793	1874	1956	2037
45	83	167	250	333	417	500	583	667	750	833	917	1000	1084	1167	1250	1333	1417	1500	1583	1667	1750	1833	1917	2000	2083
46	85	170	256	341	426	511	596	681	767	852	937	1022	1108	1193	1278	1363	1448	1533	1619	1704	1789	1874	1959	2044	2130
47	87	174	261	348	435	522	609	696	783	870	957	1044	1131	1219	1306	1393	1480	1567	1654	1740	1828	1915	2002	2089	2176
48	89	178	267	356	444	533	622	711	800	889	978	1067	1156	1244	1333	1422	1511	1600	1689	1778	1867	1956	2044	2133	2222
49	91	181	272	363	454	544	635	726	817	907	998	1089	1180	1271	1361	1452	1542	1633	1724	1814	1906	1996	2087	2178	2269
50	93	185	278	370	463	556	648	741	833	926	1019	1111	1204	1296	1389	1481	1574	1667	1759	1852	1944	2037	2130	2222	2315
51	94	189	283	378	472	567	661	756	850	944	1039	1133	1228	1322	1417	1511	1606	1700	1794	1889	1983	2078	2172	2267	2361
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53	98	196	294	392	491	589	687	785	883	981	1080	1178	1276	1374	1472	1570	1669	1767	1865	1963	2061	2159	2258	2356	2454
54	100	200	300	400	500	600	700	800	900	1000	1100	1200	1300	1400	1500	1600	1700	1800	1900	2000	2100	2200	2300	2400	2500
55	102	204	306	407	509	611	713	815	917	1018	1120	1222	1324	1426	1528	1630	1731	1833	1935	2037	2139	2241	2343	2444	2546
56	104	207	311	415	519	622	726	830	933	1037	1141	1244	1348	1452	1555	1659	1763	1867	1970	2074	2178	2281	2385	2488	2593
57	106	211	317	422	528	633	739	844	950	1055	1161	1267	1372	1478	1583	1689	1794	1900	2005	2111	2217	2322	2428	2533	2639
58	107	215	322	430	537	644	752	859	967	1074	1181	1289	1396	1504	1611	1719	1826	1933	2041	2148	2256	2363	2470	2578	2685
59	109	219	328	437	546	656	765	874	983	1092	1202	1311	1421	1530	1639	1748	1857	1967	2076	2185	2294	2404	2513	2622	2732
60	111	222	333	444	556	667	778	889	1000	1111	1222	1333	1444	1556	1667	1778	1889	2000	2111	2222	2333	2444	2556	2667	2778
61	113	226	339	452	565	678	791	904	1017	1130	1243	1356	1469	1582	1694	1807	1920	2033	2146	2259	2372	2485	2598	2711	2824
62	115	230	344	459	574	689	804	919	1033	1148	1263	1378	1493	1608	1722	1837	1952	2067	2181	2296	2411	2526	2641	2756	2870
63	117	233	350	467	583	700	817	933	1050	1167	1283	1400	1517	1633	1750	1867	1983	2100	2217	2333	2450	2567	2683	2800	2917
64	119	237	356	474	593	711	830	948	1067	1185	1304	1422	1541	1659	1778	1896	2015	2133	2252	2370	2489	2607	2726	2844	2963
65	120	241	361	481	602	722	843	963	1083	1204	1324	1444	1565	1685	1806	1926	2046	2167	2287	2407	2528	2648	2768	2888	3009
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68	126	252	378	504	630	756	882	1007	1133	1259	1385	1511	1637	1763	1889	2015	2141	2267	2393	2518	2644	2770	2896	3022	3148
69	128	256	383	511	639	767	895	1022	1150	1278	1406	1533	1661	1789	1917	2044	2172	2300	2428	2555	2683	2811	2939	3067	3194
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71	131	263	394	526	657	789	920	1052	1183	1315	1446	1578	1709	1841	1972	2104	2235	2367	2498	2629	2761	2893	3024	3156	3287
72	133	267	400	533	667	800	933	1067	1200	1333	1467	1600	1733	1867	2000	2133	2267	2400	2533	2667	2800	2933	3067	3200	3333
73	135	270	406	541	676	811	946	1081	1217	1352	1487	1622	1757	1893	2028	2163	2298	2433	2569	2704	2839	2974	3109	3244	3380
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75	139	278	417	556	694	833	972	1111	1250	1389	1528	1667	1805	1944	2083	2222	2361	2500	2639	2778	2917	3056	3195	3333	3472
76	141	281	422	563	704	844	985	1126	1267	1407	1548	1689	1830	1971	2111	2252	2393	2533	2674	2815	2956	3096	3237	3378	3519
77	143	285	428	570	713	856	998	1141	1283	1426	1569	1711	1854	1996	2139	2281	2424	2567	2709	2852	2991	3137	3280	3422	3565
78	144	289	433	578	722	867	1011	1156	1300	1444	1589	1733	1878	2022	2167	2311	2456	2600	2744	2889	3033	3178	3322	3467	3611
79	146	293	439	585	731	878	1024	1170	1317	1463	1609	1756	1902	2048	2194	2341	2487	2633	2780	2926	3072	3218	3365	3511	3657
80	148	296	444	592	741	889	1037	1185	1333	1481	1630	1778	1926	2074	2222	2370	2519	2667	2815	2963	3111	3259	3408	3556	3704
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83	154	307	461	615	769	922	1076	1230	1383	1537	1691	1844	1998	2152	2306	2459	2613	2767	2920	3074	3228	3381	3535	3688	3843
84	156	311	467	622	778	933	1088	1244	1400	1556	1711	1867	2022	2178	2333	2489	2644	2800	2955	3111	3267	3422	3578	3733	3889
85	157	315	472	630	787	944	1102	1259	1417	1574	1731	1889	2046	2204	2361	2519	2676	2833	2991	3148	3306	3463	3620	3778	3935
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88	163	326	489	652	815	978	1141	1304	1467	1630	1793	1956	2119	2282	2444	2607	2770	2933	3096	3259	3422	3585	3748	3911	4074
89	165	330	494	659	824	989	1154	1319	1483	1648	1813	1978	2143	2308	2472	2637	2802	2967	3131	3296	3461	3626	3791	3956	4120
90	167	333	500	667	833	1000	1167	1333	1500	1667	1833	2000	2167	2333	2500	2667	2833	3000	3167	3333	3500	3667	3833	4000	4167
91	169	337	506	674	843	1011	1180	1348	1517	1685	1854	2022	2191	2359	2528	2696	2865	3033	3202	3370	3539	3707	3876	4044	4213
92	170	341	511	681	852	1022	1192	1363	1533	1704	1874	2044	2215	2385	2555	2726	2896	3067	3237	3407	3578	3748	3918	4088	4259
93	172	344	516	689	861	1033	1205	1378	1550	1722	1894	2067	2239	2411	2583	2756	2928	3100	3272	3444	3617	3789	3961	4133	4306
94	174	348	522	696	870	1044	1218	1393	1567	1741	1915	2089	2263	2437	2611	2785	2959	3133	3307	3481	3656	3830	4004	4178	4352
95	176	352	528	704	880	1056	1231	1407	1583	1759	1935	2111	2287	2463	2639	2815	2991	3167	3343	3518	3694	3870	4046	4222	4398
96	178	356	533	711	889	1067	1244	1422	1600	1778	1956	2133	2311	2489	2667	2844	3022	3200	3378	3556	3733	3911	4089	4267	4444
97	180	359	539	719	898	1078	1257	1437	1617	1796	1976	2156	2335	2515	2694	2874	3054	3233	3413	3593	3772	3952	4132	4311	4491
98	181	363	544	726	907	1089	1270	1452	1633	1815	1996	2178	2359	2541	2722	2904	3085	3267	3448	3630	3811	3993	4174	4356	4537
99	183	367	550	733	917	1100	1283	1467	1650	1833	2017	2200	2384	2567	2750	2933	3117	3300	3483	3667	3850	4033	4217	4400	4583
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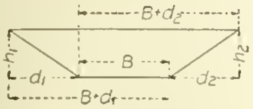
102	189	378	567	756	944	1133	1322	1511	1700	1889	2078	2267	2456	2644	2833	3022	3211	3400	3589	3778	3967	4156	4344	4533	4722
103	191	381	572	763	954	1144	1335	1526	1717	1907	2098	2289	2480	2670	2861	3052	3243	3433	3624	3815	4006	4196	4387	4578	4769
104	193	385	578	770	963	1156	1348	1541	1733	1926	2119	2311	2504	2696	2889	3081	3274	3467	3659	3852	4044	4237	4430	4622	4815
105	194	389	583	777	972	1167	1361	1556	1750	1944	2139	2333	2528	2722	2917	3111	3306	3500	3694	3889	4083	4278	4472	4667	4861
106	196	393	589	785	981	1178	1374	1570	1767	1963	2159	2356	2552	2748	2944	3141	3337	3533	3730	3926	4122	4319	4515	4711	4907
107	198	396	594	793	991	1189	1387	1585	1783	1981	2180	2378	2576	2774	2972	3170	3368	3567	3765	3963	4161	4359	4557	4756	4954
108	200	400	600	800	1000	1200	1400	1600	1800	2000	2200	2400	2600	2800	3000	3200	3400	3600	3800	4000	4200	4400	4600	4800	5000
	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	23	24	25

ESTIMATING GRADING QUANTITIES ON REVALUATION

(CONTINUED)



$$V = \frac{100}{54}\,[(B + d_1)h_2 + (B + d_2)h_1]$$

(Cubic Yards per 100 Feet)

B + d₁ or B + d₂

h₁ or h₂	26	27	28	29	30	31	32	33	34	35	36	37	38	39	40	41	42	43	44	45	46	47	48	49	50
15.	722	750	778	806	833	861	889	917	944	972	1000	1028	1056	1083	1111	1139	1167	1194	1222	1250	1278	1306	1333	1361	1389
16.	770	800	830	859	889	919	948	978	1007	1037	1067	1096	1126	1156	1185	1215	1244	1274	1304	1333	1363	1393	1422	1452	1481
17.	819	850	881	913	944	976	1007	1039	1070	1102	1133	1165	1196	1228	1259	1291	1322	1354	1385	1417	1448	1480	1511	1543	1574
18.	867	900	933	967	1000	1033	1067	1100	1133	1167	1200	1233	1267	1300	1333	1367	1400	1433	1467	1500	1533	1567	1600	1633	1667
19.	915	950	985	1020	1056	1091	1126	1161	1196	1231	1267	1302	1337	1372	1407	1443	1478	1513	1548	1583	1619	1654	1689	1724	1759
20.	963	1000	1037	1074	1111	1148	1185	1222	1259	1296	1333	1370	1407	1444	1481	1519	1556	1593	1630	1667	1704	1741	1778	1815	1852
21.	1011	1050	1089	1128	1167	1206	1244	1283	1322	1361	1400	1439	1478	1517	1556	1594	1633	1672	1711	1750	1789	1828	1867	1906	1944
22.	1059	1100	1141	1181	1222	1263	1304	1344	1385	1426	1467	1507	1548	1589	1630	1670	1711	1752	1793	1833	1874	1915	1956	1996	2037
23.	1107	1150	1193	1235	1278	1320	1363	1406	1448	1491	1533	1576	1619	1661	1704	1746	1789	1831	1874	1917	1959	2002	2044	2087	2130
24.	1156	1200	1244	1289	1333	1378	1422	1467	1511	1556	1600	1644	1689	1733	1778	1822	1867	1911	1956	2000	2044	2089	2133	2178	2222
25.	1204	1250	1296	1343	1389	1435	1481	1528	1574	1620	1667	1713	1759	1806	1852	1898	1944	1991	2037	2083	2130	2176	2222	2269	2315
26.	1252	1300	1348	1396	1444	1493	1541	1589	1637	1685	1733	1781	1830	1878	1926	1974	2022	2070	2119	2167	2215	2263	2311	2359	2407
27.	1300	1350	1400	1450	1500	1550	1600	1650	1790	1750	1800	1850	1900	1950	2000	2050	2100	2150	2200	2250	2300	2350	2400	2450	2500
28.	1348	1400	1452	1504	1556	1607	1659	1711	1763	1815	1867	1919	1970	2022	2074	2126	2178	2230	2281	2333	2385	2437	2489	2541	2593
29.	1396	1450	1504	1557	1611	1665	1719	1772	1826	1879	1933	1987	2041	2094	2148	2202	2256	2309	2363	2417	2470	2524	2578	2631	2685
30.	1444	1500	1556	1611	1667	1722	1778	1833	1889	1944	2000	2056	2111	2167	2222	2278	2333	2389	2444	2500	2556	2611	2667	2722	2778
31.	1493	1550	1607	1665	1722	1780	1837	1894	1952	2009	2067	2124	2181	2239	2296	2354	2411	2469	2526	2583	2641	2698	2756	2813	2870
32.	1541	1600	1659	1718	1778	1837	1896	1956	2015	2074	2133	2193	2252	2311	2370	2430	2489	2548	2607	2667	2726	2785	2844	2904	2963
33.	1589	1650	1711	1772	1833	1894	1956	2017	2078	2139	2200	2261	2322	2383	2444	2506	2567	2628	2689	2750	2811	2872	2933	2994	3056
34.	1637	1700	1763	1826	1889	1952	2015	2078	2141	2204	2267	2330	2393	2456	2519	2581	2644	2707	2770	2833	2896	2959	3022	3085	3148
35.	1685	1750	1815	1880	1944	2009	2074	2139	2204	2269	2333	2398	2463	2528	2593	2657	2722	2787	2852	2917	2981	3046	3111	3176	3241
36.	1733	1800	1867	1933	2000	2067	2133	2200	2267	2333	2400	2467	2533	2600	2667	2733	2800	2867	2933	3000	3067	3133	3200	3267	3333
37.	1781	1850	1919	1987	2056	2124	2193	2261	2330	2398	2467	2535	2604	2672	2741	2809	2878	2946	3015	3083	3152	3220	3289	3357	3426
38.	1830	1900	1970	2041	2111	2182	2252	2322	2393	2463	2533	2604	2674	2744	2815	2885	2956	3026	3096	3167	3237	3307	3378	3448	3519
39.	1878	1950	2022	2094	2167	2239	2311	2383	2456	2528	2600	2672	2744	2817	2889	2961	3033	3106	3178	3250	3322	3394	3467	3539	3611
40.	1926	2000	2074	2148	2222	2296	2370	2444	2519	2593	2667	2741	2815	2889	2963	3037	3111	3185	3259	3333	3407	3481	3556	3630	3704
41.	1974	2050	2126	2202	2278	2354	2430	2506	2581	2657	2733	2809	2885	2961	3037	3113	3189	3265	3341	3417	3493	3569	3644	3720	3796
42.	2022	2100	2178	2256	2333	2411	2489	2567	2644	2722	2800	2878	2956	3033	3111	3189	3267	3344	3422	3500	3578	3656	3733	3811	3889
43.	2070	2150	2230	2309	2389	2469	2548	2628	2707	2787	2867	2946	3026	3106	3185	3265	3344	3424	3504	3583	3663	3743	3822	3902	3981
44.	2119	2200	2281	2363	2444	2526	2607	2689	2770	2852	2933	3015	3096	3178	3259	3341	3422	3504	3585	3667	3748	3830	3911	3993	4074
45.	2167	2250	2333	2417	2500	2583	2667	2750	2533	2917	3000	3083	3167	3250	3333	3417	3500	3583	3667	3750	3833	3917	4000	4083	4167
46.	2215	2300	2385	2470	2556	2641	2726	2811	2896	2981	3067	3152	3237	3322	3407	3493	3578	3663	3748	3833	3918	4004	4089	4174	4259
47.	2263	2350	2437	2524	2611	2698	2785	2872	2959	3046	3133	3220	3307	3394	3481	3569	3656	3743	3830	3917	4004	4091	4178	4265	4352
48.	2311	2400	2489	2578	2667	2756	2844	2933	3022	3111	3200	3289	3378	3467	3556	3644	3733	3822	3911	4000	4089	4178	4267	4356	4444
49.	2359	2450	2541	2631	2722	2813	2904	2994	3085	3176	3267	3357	3448	3539	3630	3720	3811	3902	3993	4083	4174	4265	4356	4446	4537
50.	2407	2500	2593	2685	2778	2870	2963	3056	3148	3241	3333	3426	3519	3611	3704	3796	3889	3981	4074	4167	4259	4352	4444	4537	4630
51.	2456	2550	2644	2739	2833	2928	3022	3117	3211	3306	3400	3494	3589	3683	3778	3872	3967	4061	4156	4250	4344	4439	4533	4628	4722
52.	2504	2600	2696	2793	2889	2985	3081	3178	3274	3370	3467	3563	3659	3756	3852	3948	4044	4141	4237	4333	4430	4526	4622	4719	4815
53.	2552	2650	2748	2846	2944	3043	3141	3239	3337	3435	3533	3631	3730	3828	3926	4024	4122	4220	4319	4417	4515	4613	4711	4809	4907
54.	2600	2700	2800	2900	3000	3100	3200	3300	3400	3500	3600	3700	3800	3900	4000	4100	4200	4300	4400	4500	4600	4700	4800	4900	5000
55.	2648	2750	2852	2954	3056	3157	3259	3361	3463	3565	3667	3769	3870	3972	4074	4176	4278	4380	4481	4583	4685	4787	4889	4991	5093
56.	2696	2800	2904	3007	3111	3215	3319	3422	3526	3630	3733	3837	3941	4044	4148	4252	4356	4459	4563	4667	4770	4874	4978	5081	5185
57.	2744	2850	2956	3061	3167	3272	3378	3483	3589	3694	3800	3906	4011	4117	4222	4328	4433	4539	4644	4750	4856	4961	5067	5172	5278
58.	2793	2900	3007	3115	3222	3330	3437	3544	3652	3759	3867	3974	4081	4189	4296	4404	4511	4619	4726	4833	4941	5048	5156	5263	5370
59.	2841	2950	3059	3169	3278	3387	3496	3606	3715	3824	3933	4043	4152	4261	4370	4480	4589	4698	4807	4917	5026	5135	5244	5354	5463
60.	2889	3000	3111	3222	3333	3444	3556	3667	3778	3889	4000	4111	4222	4333	4444	4556	4667	4778	4889	5000	5111	5222	5333	5444	5556
61.	2937	3050	3163	3276	3389	3502	3615	3728	3841	3954	4067	4180	4293	4406	4519	4631	4744	4857	4970	5083	5196	5309	5422	5535	5648
62.	2985	3100	3215	3330	3444	3559	3674	3789	3904	4019	4133	4248	4363	4478	4593	4707	4822	4937	5052	5167	5281	5396	5511	5626	5741
63.	3033	3150	3267	3383	3500	3617	3733	3850	3967	4083	4200	4317	4433	4550	4667	4783	4900	5017	5133	5250	5367	5483	5600	5717	5833
64.	3081	3200	3319	3437	3556	3674	3793	3911	4030	4148	4267	4385	4504	4622	4741	4859	4978	5096	5215	5333	5452	5570	5689	5807	5926
65.	3130	3250	3370	3490	2611	3731	3852	3972	4093	4213	4333	4454	4574	4694	4815	4935	5056	5176	5296	5417	5537	5657	5778	5898	6019
66.	3178	3300	3422	3544	3667	3789	3911	4033	4156	4278	4400	4522	4644	4767	4889	5011	5133	5256	5378	5500	5622	5744	5867	5989	6111
67.	3226	3350	3474	3598	3722	3846	3970	4094	4219	4343	4467	4591	4715	4839	4963	5087	5211	5335	5459	5583	5707	5831	5956	6080	6204
68.	3274	3400	3526	3652	3778	3904	4030	4156	4281	4407	4533	4659	4785	4911	5037	5163	5289	5415	5541	5667	5793	5919	6044	6170	6296
69.	3322	3450	3578	3706	3833	3961	4089	4217	4344	4472	4600	4728	4856	4983	5111	5239	5367	5494	5622	5750	5878	6006	6133	6261	6389
70.	3370	3500	3630	3759	3889	4018	4148	4278	4407	4537	4667	4796	4926	5056	5185	5315	5444	5574	5704	5833	5963	6093	6222	6352	6481
71.	3419	3550	3681	3813	3944	4076	4207	4339	4470	4602	4733	4865	4996	5128	5259	5391	5522	5654	5785	5917	6048	6180	6311	6443	6574
72.	3467	3600	3733	3867	4000	4133	4267	4400	4533	4667	4800	4933	5067	5200	5333	5467	5600	5733	5867	6000	6133	6267	6400	6533	6667
73.	3515	3650	3785	3920	4056	4191	4326	4461	4596	4731	4867	5002	5137	5272	5407	5543	5678	5813	5948	6083	6219	6354	6489	6624	6759
74.	3563	3700	3837	3974	4111	4248	4385	4522	4659	4796	4933	5070	5207	5344	5481	5619	5756	5893	6030	6167	6304	6441	6578	6715	6852
75.	3611	3750	3889	4028	4167	4306	4444	4583	4722	4861	5000	5139	5278	5417	5556	5694	5833	5972	6111	6250	6389	6528	6667	6806	6944
76.	3659	3800	3941	4081	4222	4363	4504	4644	4785	4926	5067	5207	5348	5489	5630	5770	5911	6052	6193	6333	6474	6615	6756	6896	7037
77.	3707	3850	3993	4135	4278	4420	4563	4706	4848	4991	5133	5276	5419	5561	5704	5846	5989	6131	6274	6417	6559	6702	6844	6987	7130
78.	3756	3900	4044	4189	4333	4478	4622	4767	4911	5056	5200	5344	5489	5633	5778	5922	6067	6211	6356	6500	6644	6789	6933	7078	7222
79.	3804	3950	4096	4243	4389	4535	4681	4828	4974	5120	5267	5413	5559	5706	5852	5998	6144	6291	6437	6583	6730	6876	7022	7169	7315
80.	3852	4000	4148	4296	4444	4593	4741	4889	5037	5185	5333	5481	5630	5778	5926	6074	6222	6370	6519	6667	6815	6963	7111	7259	7407
81.	3900	4050	4200	4350	4500	4650	4800	4950	5100	5250	5400	5550	5700	5850	6000	6150	6300	6450	6600	6750	6900	7050	7200	7350	7500
82.	3948	4100	4252	4404	4556	4707	4859	5011	5163	5315	5467	5619	5770	5922	6074	6226	6378	6530	6681	6833	6985	7137	7289	7441	7593
83.	3996	4150	4304	4457	4611	4765	4919	5072	5226	5379	5533	5687	5841	5994	6148	6302	6456	6609	6763	6917	7070	7224	7378	7531	7685
84.	4044	4200	4356	4511	4667	4822	4978	5133	5289	5444	5600	5756	5911	6067	6222	6378	6533	6689	6844	7000	7156	7311	7467	7622	7778
85.	4093	4250	4407	4565	4722	4880	5037	5194	5352	5509	5667	5824	5981	6139	6296	6454	6611	6769	6926	7083	7241	7398	7556	7713	7870
86.	4141	4300	4459	4618	4778	4937	5096	5256	5415	5574	5733	5893	6052	6211	6370	6530	6689	6848	7007	7167	7326	7485	7644	7804	7963
87.	4189	4350	4511	4672	4833	4994	5156	5317	5478	5639	5800	5961	6122	6283	6444	6606	6767	6928	7089	7250	7411	7572	7733	7894	8056
88.	4237	4400	4563	4726	4889	5052	5215	5378	5541	5704	5867	6030	6193	6356	6519	6681	6844	7007	7170	7333	7496	7659	7822	7985	8148
89.	4285	4450	4615	4780	4944	5109	5274	5439	5604	5769	5933	6098	6263	6428	6593	6757	6922	7087	7252	7417	7581	7746	7911	8076	8241
90.	4333	4500	4667	4833	5000	5167	5333	5500	5667	5833	6000	6167	6333	6500	6667	6833	7000	7167	7333	7500	7667	7833	8000	8167	8333
91.	4381	4550	4719	4887	5056	5224	5393	5561	5730	5898	6067	6235	6404	6572	6741	6909	7078	7246	7415	7583	7752	7920	8089	8257	8426
92.	4430	4600	4770	4941	5111	5281	5452	5622	5793	5963	6133	6304	6474	6644	6815	6985	7156	7326	7496	7667	7837	8007	8178	8348	8519
93.	4478	4650	4822	4994	5167	5339	5511	5683	5856	6028	6200	6372	6544	6717	6889	7061	7233	7406	7578	7750	7922	8094	8267	8439	8611
94.	4526	4700	4874	5048	5222	5396	5570	5744	5919	6093	6267	6441	6615	6789	6963	7137	7311	7485	7659	7833	8007	8181	8356	8530	8704
95.	4574	4750	4926	5102	5278	5454	5630	5806	5981	6157	6333	6509	6685	6861	7037	7213	7389	7565	7741	7917	8093	8269	8444	8620	8796
96.	4622	4800	4978	5156	5333	5511	5689	5867	6044	6222	6400	6578	6756	6933	7111	7289	7467	7644	7822	8000	8178	8356	8533	8711	8889
97.	4670	4850	5030	5209	5389	5569	5748	5928	6107	6287	6467	6646	6826	7006	7185	7365	7544	7724	7904	8083	8263	8443	8622	8802	8981
98.	4719	4900	5081	5263	5444	5626	5807	5989	6170	6352	6533	6715	6896	7078	7259	7441	7622	7804	7985	8167	8348	8530	8711	8893	9074
99.	4767	4950	5133	5317	5500	5683	5867	6050	6233	6417	6600	6783	6967	7150	7333	7517	7700	7883	8067	8250	8433	8617	8800	8983	9166
100.	4815	5000	5185	5370	5556	5741	5926	6111	6296	6481	6667	6852	7037	7222	7407	7593	7778	7963	8148	8333	8519	8704	8889	9074	9259
101.	4863	5050	5237	5424	5611	5798	5985	6172	6359	6546	6733	6920	7107	7294	7481	7669	7856	8043	8230	8417	8604	8791	8978	9165	9352
102.	4911	5100	5289	5478	5667	5856	6044	6233	6422	6611	6800	6989	7178	7367	7556	7744	7933	8122	8311	8500	8689	8878	9067	9256	9444
103.	4959	5150	5341	5531	5722	5913	6104	6294	6485	6676	6867	7057	7248	7439	7630	7820	8011	8202	8393	8583	8774	8965	9156	9346	9537
104.	5007	5200	5393	5585	5778	5970	6163	6356	6548	6741	6933	7126	7319	7511	7704	7896	8089	8281	8474	8667	8859	9052	9244	9437	9630
105.	5056	5250	5444	5639	5833	6028	6222	6417	6611	6806	7000	7194	7389	7583	7778	7972	8167	8361	8556	8750	8944	9139	9333	9528	9722
106.	5104	5300	5496	5693	5889	6085	6281	6478	6674	6870	7067	7263	7459	7656	7852	8048	8244	8441	8637	8833	9030	9226	9422	9619	9815
107.	5152	5350	5548	5746	5944	6143	6341	6539	6737	6935	7133	7331	7530	7728	7926	8124	8322	8520	8719	8917	9115	9313	9511	9709	9907
108.	5200	5400	5600	5800	6000	6200	6400	6600	6800	7000	7200	7400	7600	7800	8000	8200	8400	8600	8800	9000	9200	9400	9600	9800	10000
	26	27	28	29	30	31	32	33	34	35	36	37	38	39	40	41	42	43	44	45	46	47	48	49	50

Discussion of a Case in Complicated Overhaul.

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In all the various methods of computing overhaul in earthwork, the problems are explained and illustrated by very simple cases, where the whole or a portion of a cut is hauled to make a portion or the whole of an embankment. Occasions often arise where these simple explanations are inadequate to cover complicated cases. The specific case considered in this article is well illustrative of as complicated a case as will often arise. The condition was due to the fact that material from a large waste cut was hauled an unusual distance through two smaller cuts to complete three fills, instead of obtaining material from side borrow. There being no limit of haul specified in the contract, the limit to which the material could profitably be hauled was reached only when the price for the overhaul reached the unit price for common excavation.

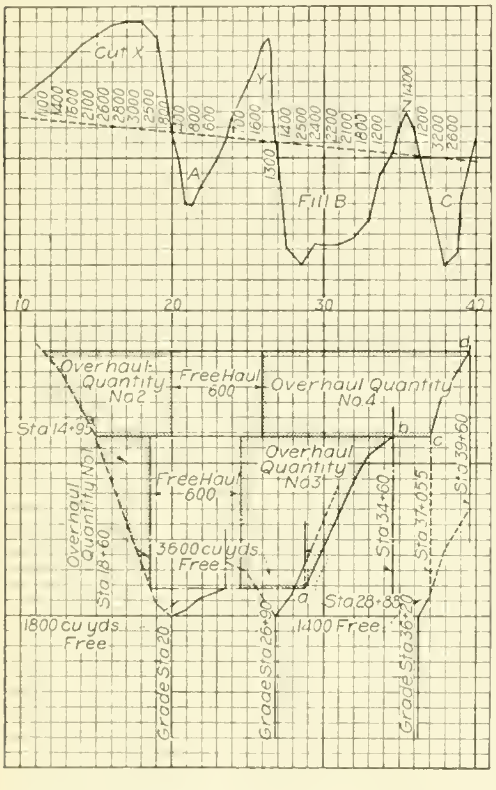


Fig. 1. Diagram Computation of a Complicated Problem in Overhaul.

The method used in figuring the overhaul is one which I believe to be the fairest to both the contractor and the railway company. However, the method of obtaining the cubic yards of overhaul is not under discussion. Other methods may be used in the figuring should they be considered more desirable and benefit be derived by the following presentation of the problem

The material from ditches in the cuts, and on the surface have been included in the excavation quantities. The material excavated being such that the actual limit of haul exceeded the station on the profile where the cross section quantities balanced, thereby denoting swell, the embankment quantities per each 100 ft. station were proportionately shrunk, according to the percentage of the total.

For the purpose of simplifying the computations and explanations the grade points are given at even pluses, while all quantities are given to the nearest 100 cu. yds.

The profile and mass diagram or overhaul chart are shown below in Fig. 1. At the top of Fig. 1 is shown the profile, the number of cubic yards of excavation and the equated embankment quantities in each 100 ft. station being indicated thereon, above and below the grade line respectively.

In the overhaul chart or mass diagram, below the profile, the 600 ft. free haul is scaled off in each case, and the pluses at which the free haul line intersects the excavation and embankment lines are noted. These pluses cause a proportionate division of the quantity within the corresponding 100-ft. section For convenience the excavation line is shown in red (see dot and dash line), while the embankment is black. It may be well to add that the quantities are platted graphically as is customary in all cases.

Beginning at the neutral grade point at Sta. 20, 1,800 cu. yds. are moved east to complete the adjoining fill *A*. Similarly the total yardage of cuts *Y* and *Z* are moved eastward. We now have 10,000 cu. yds. remaining to complete fill *B*, and 5,600 cu. yds. to complete fill *C*, all to be hauled from cut *X*. In order not to break our scale of yards on the chart, the remaining 10,000 cu. yds. in fill *B* are replatted in the new position shown by the line *a-b*. That is, the remainder of the fill line is dropped vertically until horizontal with the end of excavation in cut *X*. We now have a straight horizontal haul from *X* through *A* and *Y* to *B*. The yardage here required is readily seen by inspection to be 10,000 cu. yds.

We now replat the remainder of fill *C*, *c-d*, raising it vertically to the 11,800 cu. yd. level thereby again giving us the horizontal haul from *X*, from which the remainder of the fill is made.

The figuring for the overhaul follows in detail in Table I. For convenience the four parts are separated. Each 100 ft. section quantity is multiplied by the average haul to the free haul line. The location of this free haul within the limits of haul is immaterial, it can be located at any point desired.

TABLE I.

Overhaul No. 1—			
1,500 cu. yds. O. H.	30′	=	450 cu. y ls. O. H. 100 ft.
3,000 cu. yds. O. H.	110′	=	3,300 cu. yds. O. H. 100 ft.
2,800 cu. yds. O. H.	210′	=	5,880 cu. yds. O. H. 100 ft.
2,600 cu. yds. O. H.	310′	=	8,060 cu. yds. O. H. 100 ft.
100 cu. yds. O. H.	362.5′	=	362.5 cu. yds. O. H. 100 ft.
10,000 cu. yds.			18,052.5 cu. yds.
Overhaul No 2—			
2,000 cu. yds. O. H.	552.5′	=	11,050 cu. yds. O. H. 100 ft.
1,600 cu. yds. O. H.	650′	=	10,400 cu. yds. O. H. 100 ft.
1,400 cu. yds. O. H.	750′	=	10,500 cu. yds. O. H. 100 ft.
600 cu. yds. O. H.	827.25′	=	4,963.5 cu. yds. O. H. 100 ft.
5,600 cu. yds.			36,913.5 cu. yds.
Overhaul No. 3—			
300 cu. yds. O. H.	434′	=	1,302 cu. yds. O. H. 100 ft.
2,400 cu. yds. O. H.	490′	=	11,760 cu. yds. O. H. 100 ft.
2,200 cu. yds. O. H.	590′	=	12,980 cu. yds. O. H. 100 ft.
2,100 cu. yds. O. H.	690′	=	14,490 cu. yds. O. H. 100 ft.
1,800 cu. yds. O. H.	790′	=	14,220 cu. yds. O. H. 100 ft.
1,200 cu. yds. O. H.	905′	=	10,860 cu. yds. O. H. 100 ft.
10,000 cu. yds.			65,612 cu. yds.
Overhaul No. 4—			
3,000 cu. yds. O. H.	1,152.75′	=	34,582.5 cu. yds. O. H. 100 ft.
2,600 cu. yds. O. H.	1,280′	=	33,280 cu. yds. O. H. 100 ft.
5,600 cu. yds.			67,862.5 cu. yds.
No. 1—10,000 cu. yds. O. H.	180.525′	=	18,052.5 cu. yds. O. H. 100 ft.
No. 2— 5,600 cu. yds. O. H.	659.170′	=	36,913.5 cu. yds. O. H. 100 ft.
No. 3—10,000 cu. yds. O. H.	656.120′	=	65,612.0 cu. yds. O. H. 100 ft.
No. 4— 5,600 cu. yds. O. H.	1,211.83′	=	67,862.5 cu. yds. O. H. 100 ft.
31,200 cu. yds.			188,410.5 cu. yds.

A New Excavator of the Steam-Shovel Type for General Earth Work.

(Staff Article.)

A light portable excavator capable of being adapted to several kinds of earth moving operations is shown by the accompanying illustrations. This machine will travel at two speeds, one and three miles per hour, and is designed to go anywhere that can a team and dump wagon. For excavating three different types of scoops are provided, namely, a dipper, Fig. 1; a skimmer, Fig. 2, and a ditcher, Fig. 3, scoop. All three scoops are about equal in capacity, holding approximately two-fifths of a yard, and can be operated at about the same speed, i. e., two to three times a minute in free digging.

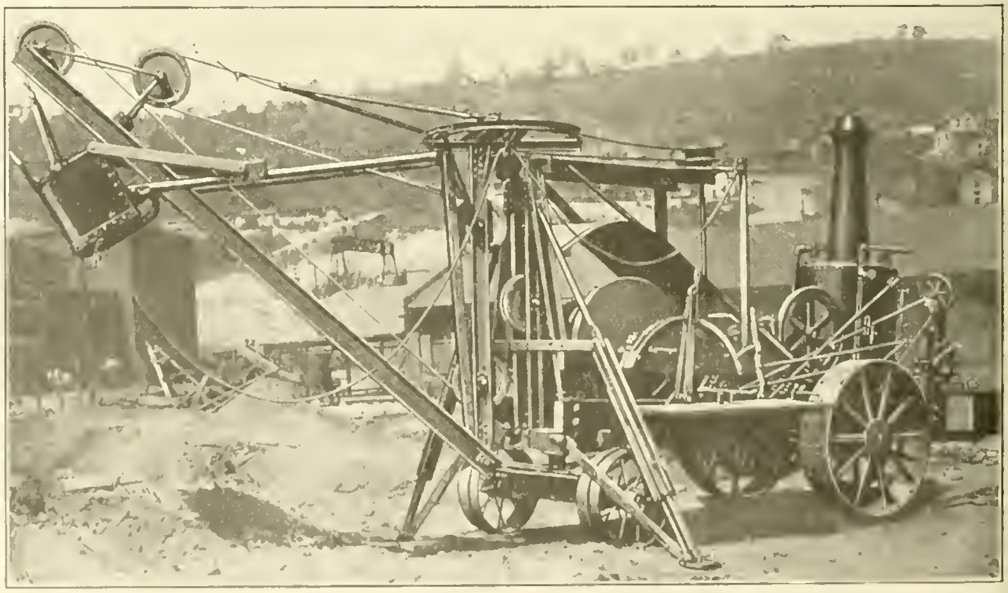


Fig. 1. Keystone Excavator Equipped With Dipper for Excavation from Bank.

The dipper scoop is carried on hinged dipper sticks as shown in the illustration, directed, lowered and raised by the boom. This boom of shovel is used for going into a side hill embankment in practically the same manner as larger steam shovels.

The skimmer scoop has a flat bottom and is used largely in street grading and for comparatively shallow excavation. It is carried on rollers which slide along the boom. When the skimmer type of shovel is to be used the dipper sticks are removed and the tackling changed. The skimmer scoop can be used advantageously for re-filling ditch. It should be noted that the form of the scoop makes it possible to have a smooth, level, finished surface in grading.

The ditching scoop differs from the dipper and skimmer in shape and is employed in making ditches for sewers, water mains, etc. It is good for a width of 15 ins. to 44 ins., and a depth of 6 or 8 ft. The best record with this type of scoop was made by S. B. Markley, contractor of Woodlawn, Pa., on work at Conway, Pa. He dug in eight hours 400 ft. of ditch 4½ to 5 ft. deep and 36 ins. wide at the bottom.

In operation the action of the dipper scoop is reversed, for ditching, the scoop being carried on a hinged arm at the extremity of the boom, and the machine being moved backward as the ditch is completed. The best record hitherto achieved with the dipper scoop was 142 loads, dump wagons of 1½ yards capacity being well filled, in 6½ hours' running time. For short periods wagons were loaded at the rate of one in each one and a quarter minutes.

Two men are required to manipulate the machine. The boiler is 36x69 ins. and is of the inverted porcupine style. The engine is 8x8 ins. The weight of the complete machine is about 16,000 lbs. The machine is made by the Keystone Driller Co., Beaver Falls, Pa.

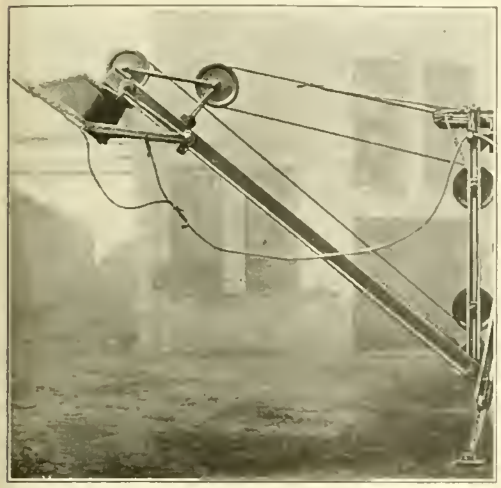


Fig. 2. Skimmer Bucket Equipment for Keystone Excavator.

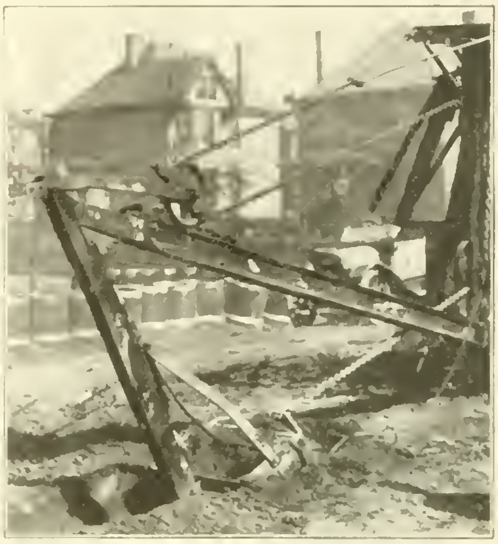


Fig. 3. Ditcher Bucket Equipment for Keystone Excavator.

Roads and Streets

Proportioning Gravel Concrete: Quantity and Cost Curves.

In proportioning the material for concrete roads and concrete waterways it is necessary that the aggregate be carefully proportioned to secure a maximum density. The importance of securing maximum density is self-evident to engineers engaged in building concrete roads, the wearing qualities of which bear a definite relation to the density of the concrete.

While clean, hard gravel is generally accepted as a superior material for concrete, there is undoubtedly a wide variation in the quality of the mixture produced. This is due in a large measure to uncertain proportioning consequent on the use of pit run aggregate. In our issue Feb. 14, 1912, we gave a method of accurately proportioning the aggregate, illustrated by various curves developed by Clifford Older, bridge engineer, Illinois Highway Commission, and the present article is a development and elaboration of this method as presented by T. R. Agg and C. S. Nichols in Bulletin 31, Engineering Experiment Station, Iowa State College of Agriculture and Mechanic Arts.

Strength.—It has been established by numerous experiments that the strength of the concrete which is made from any mixture of sand and stone depends primarily upon the ratio of sand to cement in the mixture, so long as the mortar thus made is sufficient in quantity to fill the voids in the stone. The strength of the concrete is not increased by decreasing the amount of stone below the quantity in which the voids will be just filled, nor is it materially decreased.

The exact strength of a concrete made from a mixture of sand and stone will vary considerably with the size and grading of the sand and of the stone. If the sand grains are all very small and the stone pieces all of the same size, a relatively poor concrete will be produced with a given amount of cement. If, on the other hand, the sand ranges in size from ¼ in. down and the stone ranges in size from 2½ ins. down to ¼ in., both being well graded, a relatively strong concrete can be made with a given amount of cement.

Mixtures.—The following mixtures are commonly specified for a few classes of concrete work. This will serve to indicate in a general way the work for which each is suited:

Footings on bridge abutments Brick pavement base	1-3-6, or 1-3-5
Abutments, piers and head-walls Brick pavement base, foundations	1-2½-5, or 1-2½-4
Reinforced concrete of all kinds, such as culverts, tanks, bridge floors, walls, one course pavement	1-2-4
Hand rails, girders, one course pavements, one course sidewalks, and work to take a good smooth finish	1-2-3½

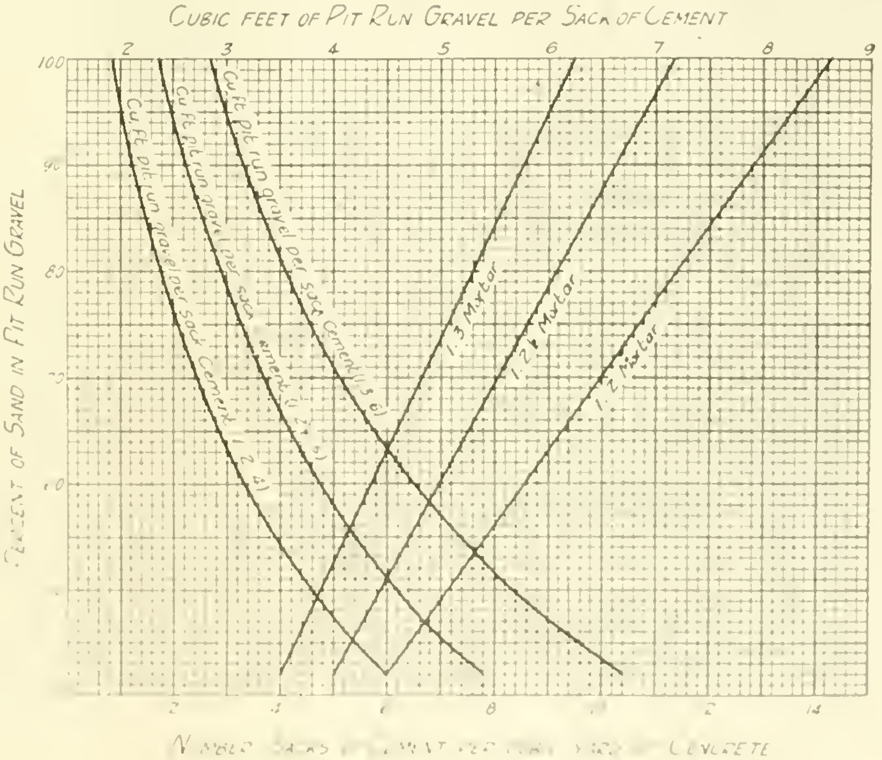


Fig. 1. Curve for Use in Proportioning Concrete.

(Based on 0.42 cu. yds. of sand 0.84 cu. yds. of stone per cubic yard of concrete.)

Definitions.—The various terms used herein are defined as follows:

Stone.—That portion of a pit run gravel that is retained on a ¼-in. screen.

Sand.—That portion of a pit run gravel that passes a ⅛-in. screen, or a ¼-in. screen, as the case may be.

Pit Run Gravel.—A mixture of sand and gravel in any proportion as it comes from the pit.

Coarse Aggregate.—Pebbles screened from pit run gravel, or crushed stone of a size that is retained on a ¼-in. screen.

Gravel.—Pit run gravel or screened and graded gravel.

Crushed Stone.—Broken stone of any character, crushed and screened over a ¼-in. screen and through a 2½-in. screen.

One Sack Cement.—A sack of cement is assumed to be 0.95 cu. ft.

Measurements are by volume measured loose, that is, shoveled or dumped into the measuring box without tamping or shaking down.

PIT RUN GRAVEL: CHARACTERISTICS AND USE.

It frequently becomes necessary to make concrete from pit run gravel and it is common practice to specify an arbitrary mixture such as 1 to 4, or 1 to 6.

A 1 to 4 concrete made from pit run gravel from one pit will differ in strength very much from a 1 to 4 mixture of pit run gravel from some other pit unless the two gravels happen to be very much alike as to the sand content.

A specification such as 1 part cement to 4 parts pit run gravel means nothing unless the allowable percentage of sand in the pit run gravel is also specified. It is generally assumed that a mixture of one part cement to four parts pit run gravel is equivalent to a mixture of one part cement to two parts sand and four parts stone. As a matter of fact they are not equivalent unless the volume of sand in the pit run gravel is just half the volume of the stone. It is apparent, then, that in making concrete from pit run gravel there must be some basis of proportioning that takes account of the varying percentages of sand in the gravel.

If the pit run gravel is to be used for making the concrete for bridge piers or for a pavement base, where a mixture of 1:2½:5 would be specified if crushed stone or screened gravel and sand were available, the amount of cement to be used with the pit run gravel would be that which would make with the sand in the gravel a 1 to 2½ mortar. The amount of stone in the gravel does not influence the amount of cement to be used, but if the volume of stone in the gravel were more than twice the volume of the sand some of the stone should be screened out. If the volume of stone in the pit run gravel is less than twice the volume of sand that does not materially decrease the strength of the concrete, so long as the ratio of sand to cement is kept constant. The amount of concrete obtained with a sack of cement in this latter case will, however, be less than if the volume of stone were just twice the volume of sand, and the amount of cement used per cubic yard of gravel would, therefore, be greater than if the sand and stone were in the proper relative proportion.

Sand.—Since the amount of sand in a pit run gravel determines the amount of cement that must be used, it becomes of considerable importance to know just what part of the mixture shall be called sand. It has become common practice to consider as sand that portion of pit run gravel that will pass through a ¼-in. screen, and most plants furnishing sand on a commercial scale have their screens arranged in that way. If, however, the matter is considered from the standpoint of scientific grading of the fine and coarse aggregates ("sand" and "stone") the maximum size of the particles of sand should vary with a variation in the maximum size of the pieces of coarse aggregate. If the coarse aggregate is graded up to 2½ inches, then ¼-in. sand would be the best, but if the coarse aggregate is graded up to 1¼ in., then sand graded from about ⅛ in. down would probably be best.

It is not necessary or desirable on small work to vary the sand screen for each deposit of gravel, even if the maximum size of stone does vary, because the grading of the material otherwise is not uniform enough to warrant such great refinement in the size of the sand.

Variation in Size of Sand.—If sand from ⅛ in. down is used it should be reasonably well graded. Just how much variation would be permissible cannot be definitely stated from the data at hand, but good results have been repeatedly obtained when the percentage of the ⅛ in. sand which passed a 1/16 in. screen was as little as 65 per cent and where it was as great as 85 per cent. With this variation it is believed that concrete may be safely proportioned on the basis of the amount of ⅛ in. sand contained.

Where the percentage of sand passing the 1/16 in. screen is less or greater than 65-85 per cent it should be used with caution and the amount of cement used based upon the amount of ¼ in. sand contained.

Pit run gravels of such a character that both the ⅛ in. and ¼ in. sands screened from them are poorly graded should not be used as gravel on work of any importance without being reproportioned or being mixed with other pit run gravels that will correct the deficiencies in their grading. Most of them are sands and could be used as such if coarse material were added.

Cleanness.—It is assumed that the material does not contain an excess of clay or loam. The percentage of clay by dry measure should not exceed 3 per cent for reinforced concrete work and 5 per cent for mass concrete, not reinforced.

If concrete is to be used for work where weight is the principal requirement and where the load in compression does not exceed 200-300 lbs. per square inch, then any pit run gravel may be safely used if the proper amount of cement is added. If, however, gravels are used in reinforced concrete where the loads in compression may reach 600-700 lbs. per square inch, much more care should be exercised and the concrete should be accurately proportioned.

Quantity of Gravel Needed for a Cubic Yard of Concrete.—The exact amount of pit run gravel necessary for one cubic yard of

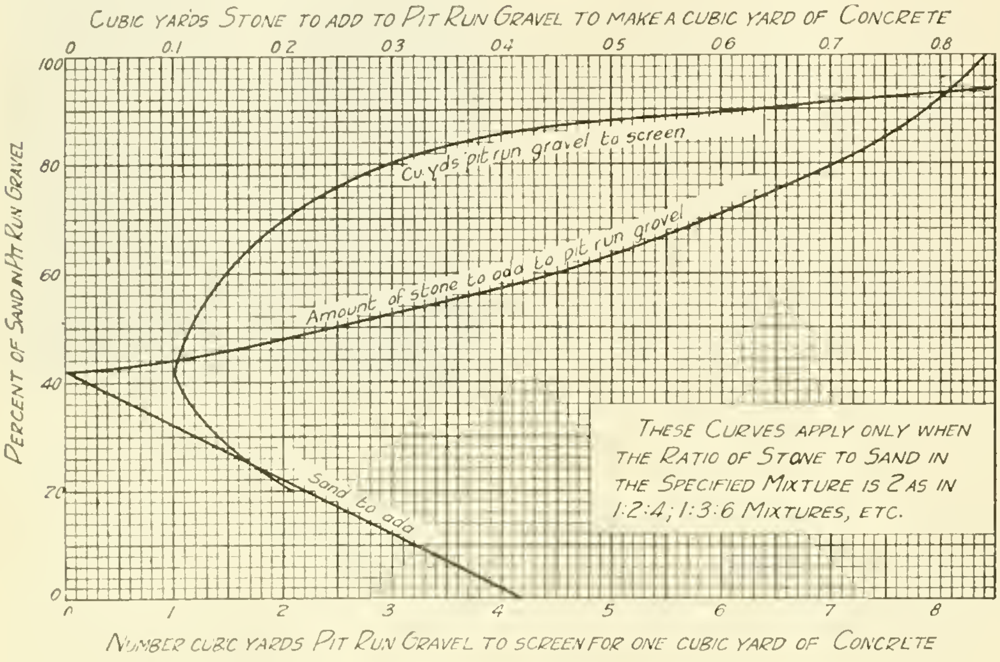


Fig. 2. Curve for Determining Correction to Make in Aggregate.

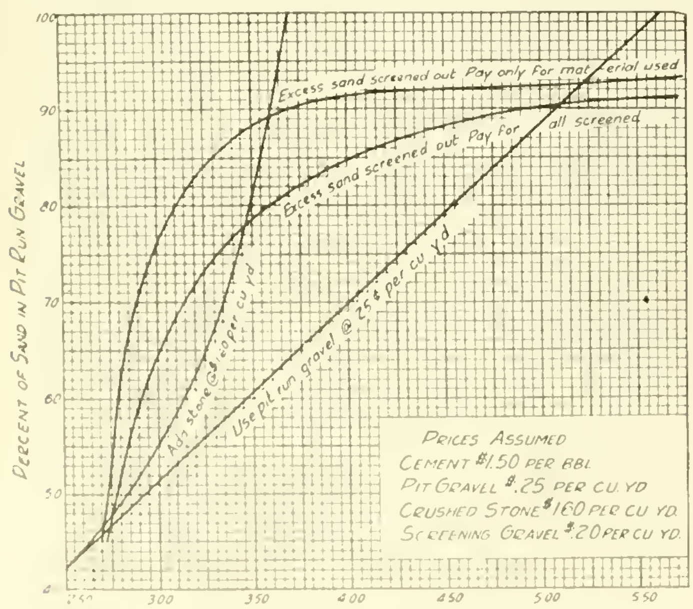


Fig. 3. Curve for Determining Cost of 1:2:4 Concrete.

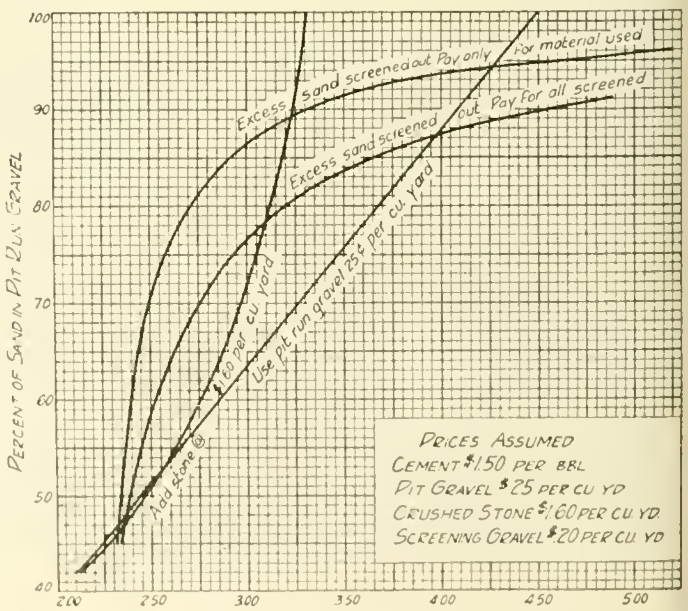


Fig. 4. Curve for Determining Cost of 1:2½:5 Concrete.

rammed concrete is hard to specify since it will depend partly upon the dryness of the gravel when measured and partly upon whether it is measured loose or packed.

It is assumed that a cubic yard of pit run gravel, loose measure, makes one cubic yard of concrete. It is also assumed that if sand and stone are measured separately and remixed for making concrete it will require 0.42 cu. yds. of sand and 0.84 cu. yds. of stone for one cubic yard of concrete. This is not exactly true for all mixtures but is nearly so for ordinary mixtures where the quantity of stone is specified as twice the quantity of sand as in 1:2:4 or 1:3:6 concrete. Curves and all computations herein are made upon this basis.

REPROPORTIONING GRAVELS: TYPICAL EXAMPLE.

As an illustration let it be assumed that concrete is to be made from pit run gravel on work for which the specifications provide a mixture of 1:2:4, but permit the use of pit run gravel if cement is added in such a quantity that the mortar meets the specifications. If the pit run gravel contains more sand than is required there are three alternatives:

1. To add cement so that the mortar will be of the specified strength, that is 1 part cement to 2 parts sand.
2. To screen the gravel and discard the surplus sand, in which case the smallest amount of cement would be used.
3. To add broken stone to the pit run gravel so that the specified ratio of sand to cement will be maintained as well as the ratio of sand to stone. In this case the same amount of cement would be used as in (2).

Which of these three methods will be followed will depend upon their relative cost and that will vary with the relative cost of the various materials and the percentage of sand in the gravel. The following analysis and discussion of a sample of pit run gravel will illustrate the manner in which the most economical method of mixing can be determined.

(1) *Cost With Pit Run Gravel.*—Let it be assumed that the pit run gravel has been analyzed and has been found to contain 60 per cent of sand and 56 per cent of stone and is to be used without reproportioning. The amount of cement to use for one cubic yard of concrete is one-half of the amount of sand in one cubic yard of the pit run gravel or ½(27x0.6)=8.1 cu. ft. Since 1 sack of cement contains 0.95 cu. ft., the number of sacks of cement to use is $\frac{8.1}{0.95}=8.52$.

Referring to Fig. 1, find the percentage of sand at the left of the diagram and follow to the right to the curve marked 1:2 mortar, and read at the bottom of the diagram the number of sacks of cement, in this case 8.52. In like manner the amount of cement may be determined for any other percentage of sand, and for any of the three mixtures given in the diagram.

When the pit run gravel is used it is desirable to know the amount of gravel in cubic feet that must be used with 1 sack of cement, so that the proper quantity can be placed in the mixer.

Since 27 cu. ft. of the above pit run gravel is to be used with 8.52 sacks of cement, the amount of gravel to use with one sack of cement is $\frac{27}{8.52}=3.16$ cubic feet. This quantity can also be determined from the diagram in Fig. 1 as follows: Find the percentage of sand at the left of the diagram and follow to the right to the curve marked "Cubic feet of pit run gravel to use with one sack of cement, 1:2:4 mixture" and read at the top of the diagram the number of cubic feet of pit run gravel, in this case 3.16. In like manner, the quantity can be determined for any other percentage of sand and for any of the three mixtures given.

If cement costs $1.50 per barrel and the pit run gravel costs 25c per cubic yard, the cost of cement and the above gravel per cubic yard of concrete is:

Cement $\frac{8.52}{4}\times\$1.50$	$3.19
Gravel	0.25
	$3.44

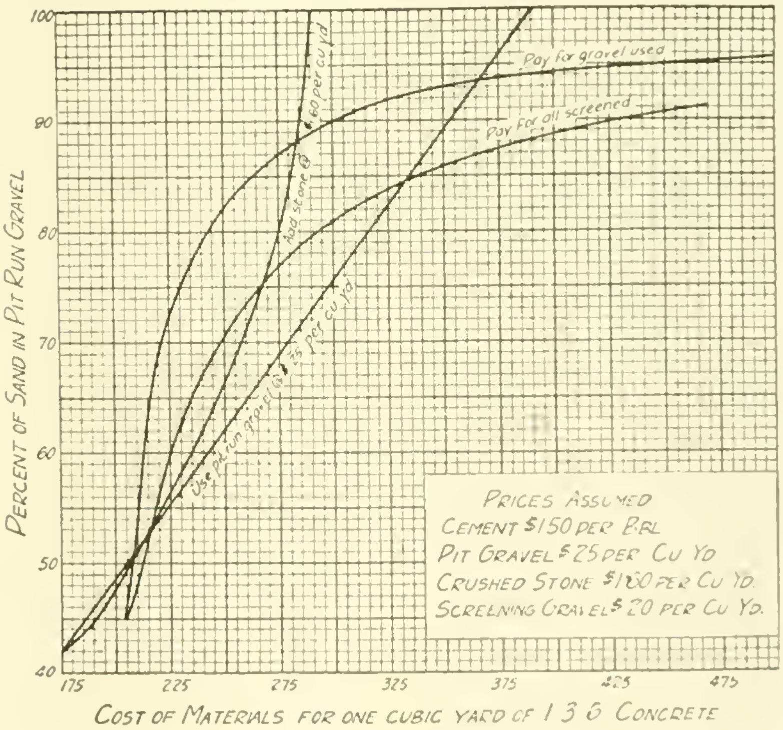


Fig. 5. Curve for Determining Cost of 1:3:6 Concrete.

The cost for cement and aggregate may also be determined from the diagram in Figure 3. At the left of the diagram find the percentage of sand which in this case is 60, and follow to the right to the curve marked "pit run gravel at 25c per cubic yard" and read at the bottom of the diagram the cost for cement and aggregate for one cubic yard of concrete, which in this case is found to be $3.44.

In the same way the cost for cement and aggregate can be determined for any other percentage of sand. If the mixture is 1:2½:5 the cost can be obtained from Fig. 4 and if 1:3:6, from Fig. 5.

(2) *Cost With Screened Gravel.*—Having the cost of cement and aggregate for one cubic yard of concrete, using the pit run gravel, find the cost of cement and aggregate for one cubic yard of concrete if the excess sand is screened out and discarded. Since the amount of sand required for one cubic yard of concrete is 0.42 cu. yds., and the amount of stone 0.84 cu. yds., the amount of the pit run gravel that must be screened to furnish enough stone for one cubic yard of concrete may be determined as follows:

The amount of stone needed is 0.84 cu. yds. and 1 cu. yd. of the pit run gravel contains 0.56 cu. yds. of stone, therefore, the amount to screen is $\frac{0.84}{0.56}=1.5$ cu. yds.

The amount to screen may also be determined from the curve in Fig. 2 as follows: Find at the left of the diagram the per cent of sand in the pit run gravel and follow to the right to the curve marked "Gravel to screen" and read at the bottom of the diagram the number of cubic yards to screen, which in this case is 1.5.

In like manner the number of cubic yards to screen can be determined for any percentage of sand. The curve holds good when the amount of coarse aggregate is specified to be twice the amount of sand, regardless of the amount of cement specified.

The cost of the aggregate for a cubic yard of concrete depends upon the cost of the pit run gravel, the cost of screening and whether the surplus sand must be paid for or may be left in the pit and not paid for. Assume that pit run gravel costs 25c per cubic yard, that the surplus sand need not be paid for and that screening costs 30c per cubic yard, and that cement costs $1.50 per barrel, and the cost of cement and aggregate for one cubic yard of concrete will be as follows:

Since 0.42 cu. yds. of sand will be used, 0.21 cu. yds. of cement will be used, or 5.67 cu. ft., and since a barrel of cement is 3.8 cu. ft., the number of barrels of cement required is $\frac{5.67}{3.8}=1.49$, or say, 1.5 barrels.

Cost of cement 1.5 × $1.50 = ...	$2.25
Cost of 1 cu. yd. pit run gravel..................	0.25
Cost of screening 1.5 cu. yd..................	0.45
Cost of aggregate per cu. yd.............	$2.95

If the surplus sand must be paid for it will add to the above the cost of 0.5 cu. yd. sand discarded, or 12½c, making the cost of the aggregate for one cubic yard of concrete $3.07.

The cost of aggregates for a cubic yard of concrete when the percentage of sand is known and when the mixture is 1:2:4, 1:2½:5 or 1:3:6 can be read from the diagram in Figs. 3, 4 and 5.

These diagrams also give a curve of costs when all material screened must be paid for and are used in the same manner as the curves already described which are given in Fig. 3.

Cost When Stone is Added.—If crushed stone is available, it may be cheaper to add it to make up the deficiency in coarse material than to screen the gravel. To determine the quantity of stone to add for one cubic yard of concrete, the amount of the pit run gravel to use to furnish the proper quantity of sand must first be determined. Since it requires 0.42 cu. yd. of sand for a cubic yard of concrete and each cubic yard of the pit run gravel contains 0.6 cu. yd. of sand, the quantity of the pit run gravel needed to furnish sand for a cubic yard of concrete is $\frac{0.42}{0.6}=0.7$ cu. yd.

Since the pit run gravel contains 56 per cent of stone, there will be in the 0.7 cu. yds. of pit run gravel 0.7 × 0.56 or 0.392 cu. yd. of stone, and since 0.84 cu. yd. of stone is needed for a cubic yard of concrete it will be necessary to add to the 0.7 cu. yd. of pit run gravel 0.84−0.392=0.45 cu. yd. of stone. The cost of the materials for one cubic yard of concrete will, in this case, be as follows, assuming the stone to cost $1.60 per cubic yard:

Cost of pit run gravel, 0.7 × 0.25 =	$0.17
Cost of stone to add, 0.45 × $1.60	0.72
Cost of cement, 1.5 × $1.50	2.25
Cost material for 1 cu. yd. of concrete	$3.14

The amount of stone to add to pit run gravel containing any percentage of sand, to give material for one cubic yard of concrete, can be obtained from Fig. 2 by finding the percentage of sand at the left of the diagram and following to the right to the curve marked "Amount of stone to add to pit run gravel" and reading the quantity to add at the top of the diagram. It should be noted that if the sand content of the pit run gravel is less than 42 per cent, then sand must be added as indicated in the diagram or some of the stone must be screened out of the pit run gravel and wasted.

TABLE I.—METHOD OF TABULATING ANALYSIS OF VARIOUS IOWA GRAVELS.

Sample number.	Percentage of stone (above 1-8 in.).	Percentage of sand (passing 1-8 in.).	Percentage of sand passing 1-16 in. screen.	Ratio of sand to stone.	Voids.		Percentage of clay.	Cement per cu. yd. concrete, using pit run gravel.			Cement per cu. yd. concrete, using screened and reproportioned gravel.			Cubic yards pit run gravel to screen to secure material for 1 cu. yd. concrete.	Cubic yards stone or sand to add to provide materials for 1 cu. yd. concrete.	Location of pit.
					Sand.	Stone.		Sacks.	Barrels.	Cost $1.50 bbl.	Sacks.	Barrels.	Cost $1.50 bbl.			
1	16	84	81	5.25	35	38	1.71	12.00	3.00	4.50	6.00	1.50	2.25	3.75	0.728	Maquoketa.
6	8	92	75	11.50	40	41	3.60	13.00	3.25	4.90	6.00	1.50	2.25	7.50	0.789	Red Oak.
8	12	88	78	7.33	34	34	1.69	12.50	3.13	4.70	6.00	1.50	2.25	5.00	0.760	Pacific Junction.
20	22	78	61	3.54	29	42	7.82	11.00	2.75	4.12	6.00	1.50	2.25	2.73	0.674	Sibley.
28	45	55	63	1.22	30	39	0.63	7.80	1.95	2.93	6.00	1.50	2.25	1.34	0.360	Correct'ville, Fleming Pit.
29	28	72	57	2.57	35	35	0.19	10.25	2.56	3.84	6.60	1.50	2.25	2.15	0.611	Ottumwa, river gravel.
4	8	92	87	11.50	33	38	2.32	13.10	3.28	4.92	6.00	1.50	2.25	7.50	0.789	Le Mars, Kramer Pit.
5	33	67	88	2.03	42	46	10.24	9.50	2.38	3.57	6.00	1.50	2.25	1.82	0.576	Clemont, Williams Pit.
9	5	95	93	19.00	37	44	1.93	13.50	3.38	5.07	6.00	1.50	2.25	12.00	0.809	Clarinda.

NOTE—It should be noted that the per cent of sand and stone has been determined by weight and not by volume, and for purposes of comparison the percentage of sand should be used. The percentage of stone is not comparable with results of test made by volume measure.

Summary.—Now, having determined the cost of the aggregate for one cubic yard of concrete by each of the three methods of proportioning the pit run gravel under discussion, the most economical can be chosen. For convenience the three possibilities are summarized as follows:

Cost Per Cubic Yard of Concrete for Aggregate and Cement.

1. Use without reproportioning	$3.44
2. (a) Screening out and discarding surplus sand and paying only for gravel used	2.95
(b) Screening out and discarding surplus sand, but paying for all gravel used	3.07
3. Adding crushed stone	3.14

It will be seen that the cheapest method is to screen out and discard surplus sand, if only material used is paid for. If all gravel screened must be paid for it is still cheaper than to add stone, although the difference is slight. It is by far the most expensive to use the pit run gravel without reproportioning.

CONCLUSIONS.

The relative costs of the three methods outlined will vary with the costs of materials and the cost of labor, and for any given set of conditions these costs must be worked out before a decision can be reached, as to the most economical procedure. The quantities of materials can readily be determined from curves shown in Figs. 1 and 2 and the costs can then be determined when the price is known.

1:2:4 Mix.—A study of Fig. 3 which applies to concrete mixed in the proportions of 1:2:4 and with materials at the prices indicated on the diagram, shows that if the sand content of the gravel is more than 50 per cent and less than 95 per cent it pays to screen and reproportion. It also shows that it is much cheaper to add crushed stone than to use the pit run gravel. If the sand is above 80 per cent it is cheaper to add stone than to screen if the surplus sand must be paid for.

1:2½:5 Mix.—In the same way it can be seen from Fig. 4 that for 1:2½:5 concrete with materials at the prices indicated on the diagram, if the percentage of sand is less than 50 per cent there is no great saving to be obtained by reproportioning, but if the percentage of sand is between 50 and 95 it will be a decided saving to screen. The diagram also shows that if all the material screened must be paid for, including the surplus sand, then if the sand content is greater than 75 per cent it will be cheaper to add stone than to screen the gravel.

1:3:6 Mix.—Similarly, Fig. 5 shows that for a 1:3:6 concrete with materials at the prices indicated on this diagram, it pays to screen, if the sand content exceeds 55 per cent and is less than 93 per cent. It also shows that if all material screened must be paid for it is cheaper to add stone than to screen if the sand content exceeds 75 per cent, and if only the material used is paid for it is cheaper to add stone than to screen, when the sand content exceeds 88 per cent.

These relations hold true whether ⅛ in. or ¼ in. sand is used and the decision as to which size will be adopted for sand depends upon the size of the coarse aggregate as previously explained.

If the ¼ in. sand is used more cement will be required with pit run gravel than if ⅛ in. sand is used, but in case of doubt, use the ¼ in. sand.

It should be noted that the typical cases worked out in the preceding pages give results which are comparable only when the prices paid for pit run gravel, crushed stone, screening and cement are as indicated. For other prices the quantities can be determined from Figs. 1 and 2, the cost being computed as outlined in the illustrative example. The computations are simple and the tests can readily be made in the field and adjustments in the methods of proportioning made in accordance with the tests.

In this connection it should be noted that for concrete roads or pavements the pit run gravel should, in every case, be screened and the sand and stone thus obtained used for the concrete. Pit run gravel should never be used for concrete road or pavement construction.

Summary of Suggestions Offered at the Philadelphia Snow Removal Conference.

The methods and costs of snow removal as demonstrated by present practice in a number of representative cities, have been presented in articles on snow removal which appeared in our issues of April 22, 29, May 6, 13 and 20. In this article Mr. Morris R. Sherrerd, Chief Engineer for the Board of Street and Water Commissioners, Newark, N. J., briefly sums up the suggestions made at the Philadelphia Snow Removal Conference.

It may be of advantage briefly to sum up some of the suggestions proposed: (1) The study of the problem and its relation to local conditions in advance of the winter season. (2) The investigations of proper dumping places and the preparation of special manholes and branch sewers. (3) The consideration of what sewers are adapted for the purpose of receiving snow. (4) The organization of forces for supervision and inspection. (5) The arrangement in advance with local contractors for securing carts, wagons and men on short notice. (6) The method of pay for such outside assistance, either by contracts in advance or upon yardage or day method of payment. (7) The concentration on this work at short notice of all available city employes from every department of the city's activities from which men available for the work can be spared.

The seriousness of the problem is evidently one of getting satisfactory results in the shortest possible time, and from an engineering standpoint, all studies should be directed to this end. This will require constant special preparation, concentration and distribution of other forces available to obtain the best and most economical results. These results should be the restoration of normal traffic conditions as soon as possible after a snow storm which is of sufficient magnitude to interfere with this traffic.

Co-operation With Traction Company.—The restoration of traffic applies equally, and perhaps more forcibly, to the maintenance of trolley traffic as to provision for other forms of street traffic. In the larger cities the trolley companies have generally made adequate provision to take care of the removal of snow from their tracks, but it is often done at the expense of the use of streets on the side of the trolley lines for the storing of snow removed from the tracks. Here again the element of co-operation must be brought into play. In a very heavy snowfall there is a great demand for laborers and teams for the purpose of snow removal, and it has been the experience of most of the trolley companies that practically all of the men which they can obtain are needed to keep their lines in working condition.

One method of co-operation between the trolley companies and the municipality which has worked more or less satisfactorily, has been the division of work between the trolley company and the city by which the trolley company not only removes the windrow on certain designated streets which are agreed upon in advance, but removes all of the snow from curb to curb on these designated streets, the city on its part undertaking to do the same thing on an equivalent mileage, so that the available force of both the trolley company and the city are immediately at work clearing up congestion due to a snowfall, and are not conflicting with each other.

Hauling in Gondola Cars.—In this arrangement it has been found of advantage to have the trolley company assume the clearing of the narrow streets in the busy section of the city through which their tracks run. This work can be done more advantageously by them, particularly at night, by the use of gondola cars on the street railway tracks. For the convenience of the traveling public who may use the trolley cars, it is important for the city to provide a space on the side of the track where other traffic may readily move, otherwise the invariable blocking of street cars will result from the use of the tracks by heavily loaded trucks.

Snow Plows.—To quickly obtain this result more attention might properly be given by the traction companies to some form of snowplow which would throw or scrape the snow removed from the tracks to a further distance than is now the practice, getting the windrow sufficiently far from the tracks to give a space for one vehicle. This will result on the wider streets, where it can be advantageously done, in immediately keeping open for vehicular traffic such wide streets on which the trolleys run, and also keep the vehicular traffic from the necessity of using the tracks. This work can be done by the trolley company concurrently with the operation of their sweepers and this additional scraping thus being undertaken at short intervals of time during the snowfall, so that the amount of snow to be so removed on each trip would be limited to say from one to two inches in depth and while the snow is more or less easily removed than is the case with the windrow which is now thrown up by the usual practice employed by the railway companies, this also brings into play a very potent factor in snow removal, which would be an additional help without resorting to the necessity for the employment of extra labor.

The operation of this side-extended snow plow (if we might so designate it) during the early periods of a snowfall and before traffic has to any serious degree packed the snow on the side of the tracks and also before the snow from the sidewalks has been shoveled into the street will, it is believed, make it possible to obtain satisfactory results from such a method that could not be accomplished if it were attempted to be done after the snow had ceased falling.

Track Sweepers.—It might be possible to go a step further along the lines suggested by having special track sweepers operated by trolley power which would extend over the sides of the track having the same arrangement, with a hinged joint, as is now done with trolley sprinklers, so that they could be dropped back parallel to the side of the car when passing through narrow streets or meeting an obstruction. It is the writer's intention to endeavor to obtain the operation by the street railway company of some such equipment to get the advantage of the further removal of the windrow from the tracks, as a substantial compliance with the franchise ordinance requirements mentioned above, namely, that this snow shall be removed within 24 hours after a snowfall. But it is believed that any other municipality where this obligation does not rest upon the traction company for snow removal, may obtain the advantage of the use of some such additional equipment by the railway companies, and that such an arrangement would be of sufficient benefit to justify a contract being made between the city and the railway company for this purpose.

Flushing.—The discussion in regard to flushing snow through the sewers brings to the writer's mind a factor gained from his own experience in the operation of water works, which may be of advantage in increasing the capacity of the sewer system, and which has not been given due weight in this discussion.

In the operation of a long line of water conduit on a gravity supply, during a severe winter, the formation of anchor ice in the intake reservoir was so serious as to interfere with the normal flow of water through the conduit. During the second day it was found that the water delivered into the city reservoirs was very dirty and not in its usual good condition. Several days afterwards a comparison of the meter readings as to the flow of water through this conduit before and after this incident in connection with the anchor ice showed that the conduit whose former maximum capacity was 36 million gallons per 24 hours had, for some reason, increased to 37¼ million gallons in 24 hours, and the only satisfactory way to account for this increase of carrying capacity seems to be that the anchor ice had scraped off the slime from the interior of the pipe and thereby reduced the frictional resistance on the flow of water; so it may be that the unwarranted accumulation of snow on the streets, is in a measure, an advantage to the sewer system which has not so far been appreciated.

The Committee on Water and Ordinances of the Wheeling, W. Va., City Council has received and taken up for consideration the report of the engineers employed by the city to investigate and recommend a system of water filtration. The engineers reported in favor of a rapid sand or mechanical plant as best adapted to the city.

Bituminous Filler and Dry Mortar Bed in Creosoted Wood Block Pavement Construction.

A wood block pavement is in a large measure a standardized structure. Its efficacy to meet certain traffic conditions is well established. The discussion of its shortcomings and present methods of overcoming them which is given here was prepared from a paper by R. S. Manley, president of the Creosoted Wood Block Paving Co., New Orleans, La., at the tenth meeting of the American Wood Preservers' Association, and the discussion thereof by H. L. Collier.

A creosoted wood block pavement should show no evidences of wear for many years if the proper materials are used, and if they are assembled in the proper way. The correct depth of base, or foundation, varies with the soil conditions, but the materials forming this concrete foundation and the methods of mixing are in such common use as to be standard and easily secured.

The principal causes of defects in the wood surface of more or less serious nature are (1) irregular or uneven surface due to careless laying, shifting of sand cushion, or breaking or settling of concrete, and (2) expansion difficulies due to the entrance of water into the blocks either by way of the joints or from below.

An irregular or uneven surface is death to any paving material because a depression in the surface holds water and repeated churnings of wagon wheels in the depression are bound to cause an enlargement and deepening of the depression. To avoid, the concrete should be mixed quite wet and finished smoothly with a flat wooden spreader, which gives a surface practically as even and uniform as could be obtained with a templet. On this spread ½ in. to 1 in. of clean sand, making the sand cushion conform to the contour of the finished street. Place the blocks quite closely together, roll thoroughly until a perfect surface with no inequalities has been obtained and until the blocks are firmly in place. It will require a great deal of rolling to accomplish this, but the end justifies the means. After this fill all joints two-thirds full of hot bituminous filler of such melting point as is suited to climatic conditions, and spread a thin coating of sand thereon. The use of the bituminous filler is, in my estimation, most important. It converts the street into an effective watershed which, without absorbing any of the water, directs it into storm sewers or other drainage paths. Should any water remain on the surface the wind and the sun, both good evaporative agencies, will rapidly dissipate it.

Expansion difficulties are eliminated by the use of bituminous filler, for there can be no expansion without absorption of water, and no absorption of water when all rainfall is conducted quickly to drainage sewers. In addition to this it must be remembered that with the bituminous filler each block is surrounded by an individual expansion joint.

Another way of constructing wood block surface which is sometimes recommended is to provide a mixed sand and cement cushion and sand filled joints or interstices. The sand and cement cushion does not give the opportunity for absolutely smooth surface that the sand cushion gives and is considerably more costly. The sand filler in the joints allows moisture to be absorbed in the pavement, and ultimately this moisture gets into the blocks and trouble ensues. It is only on extremely heavy traffic streets that sand can be used as a filler without expecting some expansion difficulties sooner or later. The proof of the pudding is the eating and the proof of theories of wood block construction lie in the actual occurrences on the street.

It can be stated without fear of successful contradiction that every sand-filled pavement in the South has at one time or other given trouble from uncompensated expansion, and with equal confidence it can be stated that not one bituminous-filled pavement has given trouble from this cause.

There have been objections put forward to the bituminous filler because of the belief that it would produce a sticky surface, disagreeable in warm weather; but if the proper filler is secured, and it is correctly applied there can be no such objection. The suitable filler has a consistency of rubber and can be taken in the fingers, bent and twisted without soiling the fingers. In applying this filler a spreader with squegee attachment places the filler in the joints where it is needed and not on the surface of the blocks where it is not needed.

It is proper also to use less creosote oil per cubic foot of timber when bituminous filler is used for the primary function of the creosote oil in this case is to preserve against decay instead of trying to make the creosote oil fill the double role of preservative and absolute waterproofing. No one familiar with preservative methods and history will question the efficacy of 16 pounds of creosote oil per cubic foot in preserving against decay for an indefinite period. We, therefore, see that bituminous filler can be used carefully and without inconvenience because of stickiness.

To sum up, therefore, provide adequate smooth concrete foundation, use enough sand to cover any inequalities in the concrete or depth of blocks (except in railway areas and on grades, where use sand and cement mixed), lay blocks tightly, roll until smooth, fill joints with bituminous filler, spread coating of sand, and turn on traffic.

A sand and cement cushion is mixed dry in the proportion of one part of cement and three parts of sand, struck-off with a templet and the blocks set thereon grain vertical close jointed sidewise and with a ⅛ in. joint between the ends of the blocks. The concrete base is given three or four days in which to harden, and when sufficiently strong to work over the sand and cement mixture above described placed on the same to take up any irregularity in the surface of the concrete and in the length of the blocks. There is sufficient moisture in the concrete base to bring about chemical action between the sand and cement in less than twelve hours. This cushion can be rolled as readily and as satisfactorily as the pure sand cushion. An advantage this cushion has over sand cushion, is that when chemical action has taken place, the blocks having been tamped to a firm bearing on same before hardening, it affords a perfectly smooth support for all parts of the block, and even though water should get into the pavement either from an opening between the blocks or from a broken water pipe, this foundation cannot be disarranged. There is always a certain amount of vibration in a wood block pavement under traffic, and should the sand cushion get dry, this vibration would cause the sand to trickle into the honeycombs of the concrete base to such an extent that there would be formed on the surface of the pavement slight saucerholes, which would not be the case if the cushion was made of sand and cement.

The old London method of laying the pavement in a paving pitch wash was not a bad one; in fact, if the concrete was finished with a smooth surface exactly parallel with the finished surface of the pavement and hot paving pitch, with 10 per cent by weight of Portland cement added, and spread with a squegee into which the blocks are set promptly, grain vertical, no construction will give better results.

Under no condition should less than 16 lbs. of oil to the cubic foot of timber be used and the blocks invariably placed on a ½ in. cushion of sand and cement mixed in the proportion of 1:3 and a bituminous filler so fluxed as not to melt in summer or become brittle above 15 degrees Fahr., applied when heated to at least 300 degrees and worked back and forth rapidly with a squegee until all the cracks and interstices are filled. A pavement so constructed will prove entirely satisfactory. This is the record made to date. To emphasize the necessity of the use of a bituminous filler, a treatment of 16 lbs. with a bituminous filler is preferable to a treatment of 20 pounds with a sand filler. One half inch is recommended as to the depth of the sand-cement cushion, because if the contractor is required to finish his concrete foundation with a templet to within a ½ in. of the finished surface, he will do better concrete work than if allowed to complete the same with an irregular surface.

As to the particular oil to be used, we need a pure oil, one that will preserve the timber, and at the same time have specific gravity sufficiently great to help make non-absorbent the blocks, say not far from 1.08 specific gravity. A proper oil and an elastic filler will prevent buckling, bleeding and slipperiness.

Notes on Macadam Maintenance in York County, Ontario.

Canadian practice in maintaining macadam roads is very similar to that in the United States. A brief discussion of the methods used in York County, Ontario, is given here as presented by E. A. James, consulting engineer for the York County Highway Commission in a paper before the recent Road Congress at Montreal.

There are many agencies tending toward the destruction of macadam roads, such as the grinding and loosening action of horse drawn or motor vehicles, the scouring and softening effect of water, and the action of wind in carrying off the metal in the form of dust—natural or due to fast moving vehicles. Much may also be said of the detrimental effects which result from bad drainage, narrow tires carrying heavy loads, the tracking of vehicles and the presence of foreign materials on the road, such as mud, etc., which cause wheels to pick up the metal.

Proper construction is an absolute necessity if maintenance charges are to be kept within reasonable limits. The subgrade must be well bound, the shoulders properly rounded and compacted, and proper drainage provided and maintained. Ditches should be kept open at the sides, and the crown and shoulder kept in such shape that the ditches can perform their proper function.

In construction perhaps too little attention is paid to minimum grades. The formation of ruts prevents lateral drainage, and a small quantity of water standing on the surface of a macadam road causes a local softening of the road metal, so that shallow ruts soon become furrows. This trouble insomuch as due to the softening effect of water, may be prevented by providing longitudinal as well as lateral drainage.

The average 12 ton roller carrying coal and water has approximately 18,000 lbs. on the rear axle. The rear roller width is usually 20 ins., and there is a pressure of 450 lbs. per inch on the width in contact with the road. In comparison, an average load of 2 cu. yds. of crushed stone weighing with the wagon 6,600 pounds, or 1,650 lbs. per wheel, gives for a 2-in. tire, 825 lbs. per inch of width. It will be seen that a 4-in. tire, giving a pressure of 412 lbs. per inch of width is approximately equivalent to a 12-ton roller.

TABLE I.—COST PER APPLICATION OF OILING ONE MILE OF ROAD, OF DIFFERENT WIDTHS WITH VARYING QUANTITIES OF MATERIAL, IN YORK COUNTY, ONTARIO.

Oil, at 7½ cts. per gallon. Cost of application. $25 per mile.

Width of road, ft.	Square yards per mile.	¼ gal. per square yard.	½ gal. per square yard.	¾ gal. per square yard.	1 gal per square yard.	1¼ gals. per square yard.
10	5,867	$135	$245	$355	$465	$ 575
12	7,040	157	289	421	553	685
14	8,213	179	333	487	641	795
16	9,387	201	377	551	729	905
18	10,560	223	421	619	817	1,015
20	11,733	245	465	685	905	1,125

Oil, at 8¼ cts per gallon. Cost of application, $25 per mile.

Width of road, ft.	Square yards per mile.	¼ gal. per square yard.	½ gal. per square yard.	¾ gal. per square yard.	1 gal. per square yard.	1¼ gals. per square yard.
10	5,867	$150	$275	$400	$ 525	$ 650
12	7,040	175	325	475	625	775
14	8,213	200	375	550	725	900
16	9,387	225	425	625	825	1,150
18	10,560	250	475	700	925	1,150
20	11,733	275	525	775	1,025	1,275

The carrying of mud, etc., upon the road may be prevented by keeping the shoulder and side roads smooth and well compacted by dragging.

While oiling cannot be considered a permanent remedy, it goes far towards preventing the carrying away of the metal in form of dust, as well as assisting in making the road surface waterproof. Table 1 gives an idea of the cost of oiling in York County. This cost includes heating the oil.

Ordinarily the following applications will suffice: First year—First application, ½ gal. per square yard; second application, ¼ gal. per square yard. Subsequent years—Two applications, each ¼ gal. per square yard.

There are three general systems of maintenance, viz: (1) contract system, (2) personal supervision by rural population, i. e., statute labor system, and (3) patrol system.

The contract system is not satisfactory, it being practically impossible to get proper observance of the term of contract. The statute labor system is unsound in principle, unjust in its operation, wasteful in its practice and generally unsatisfactory in results.

The patrol system is the most popular and best method of macadam road supervision. A man is given charge of a section of road from 4 to 6 miles in length, for the maintenance of which he is held personally responsible to the road authorities. Extensive tools are not necessary, a wheelbarrow, a shovel and a tamping iron comprises the outfit. With these tools ruts and holes are repaired in their infancy. The system should be put in operation as soon as possible after the completion of construction.

The best method of repair is to sweep the hole or depression clean and paint with a light tar, throwing into the hole enough 1-in. stone to bring the depression to the form of the road, adding a little more tar, some ½-in. chips and rolling until firm. Repairs are made without disturbing the base. When tar or asphalt are not available pick the stones loose for a few inches around the depression, add enough fresh material to fill, wet and tamp firmly. In a day or so the patch may have loosened slightly but if gone over a few times it will soon set as sound as the surrounding road.

A method of more extensive repair which has been tried with considerable success on heavily traveled roads by the York Highway Commission, consists of resurfacing macadam with bituminous material. Ruts were coated with ⅛ in. of Tarvia and into this was rolled enough stone to bring the road to the proper cross section. The surface was then given an application of about ½ gal. of Tarvia per square yard, into which was rolled rock chips or gravel sufficient to prevent bleeding. Granite, trap rock and pea gravel have been used, but trap rock on account of its greater hardness and toughness is undoubtedly best. A bituminous surface of this description is much more resilient than macadam. The cost is from $600 to $800 per mile of 16-ft. road, depending upon the condition of the base.

Methods of Making Field Tests of Sand-Clay Mixtures.

The determination of the qualities of different sand-clay mixtures is largely a matter of field experiment. Results of tests so far devised are, in a measure, of use only to the individual making the test. No satisfactory standard of comparison has as yet been discovered although the work of Mr. Jno. C. Koch of the University of Georgia, which was described in full in the issue of ENGINEERING AND CONTRACTING for March 18, 1914, is interesting and represents an advance in this direction.

In the Proceedings American Society Civil Engineers, pp. 1084-5, there are described by E. W. James tests for determining the correct percentage of sand and the slaking qualities of the clay which have been in vogue among sand-clay road builders for a number of years. These tests are of value not only in preliminary investigation but in determining correct proportions during construction and after completion of the work. The tests as systematized by Mr. James are given here.

The sources of material are located and selection is made, depending usually on the length of haul and the depth of the deposits beneath the surface. The immediate problem is to determine the best mixture of the materials selected.

Typical samples are taken of both sand and clay. Mixtures are made, ranging from 1 part sand to 3 parts clay, up to 3 parts sand to 1 part clay, or sometimes beyond these limits, if the materials appear to warrant it. These mixtures should be made to vary by one-half of 1 part, 1:3, 1:2½, 1:2, etc., 1½:1, 2:1, 2½:1, etc., and should be worked up with water into putty-like masses.

From each test, mix a small sample of from 1 to 2 cu. ins. cut out with a small measure. A small medicine glass, or even a large brass thimble is handy. It is only essential to get equal samples from each test mix. These samples are then rolled between the palms of the hands into reasonably true spheres and placed in the sun to dry. Some designating marks may be scratched on them.

When thoroughly baked, they are placed in a circle in a flat pan or dish, and enough water is poured into the pan to cover them, care being taken not to pour the water on the samples. Slaking will begin at once and will proceed at different rates. The sandy specimens will break down first, those with excessive clay will disintegrate second in order, and those having about the proper proportions will act more slowly. Usually, there will be one or two that determine the proper proportions of the materials, and, in the writer's experience, these will usually lie together in the series of test mixtures.

A supplementary test of some value can also be made on the dry spheres. Lightly rubbed with the thumb, those having too much sand will break down rapidly. Those having too much clay will soon begin to "dust" away, while those having the most stable mixtures will assume a slightly glazed effect under the light rubbing, due to the moisture and oil of the skin.

These two tests will not give the same results. The dry test will indicate a mixture richer in clay as the better one, and the wet test will indicate a sandier mixture. The sample indicated as satisfactory under the wet test that lies between the other two will prove best in service. These tests do not determine the absolute sand or clay content of the mixture, for the clay selected almost always contains sand, and this, in most cases, contains silt or clay. The tests serve, however, to fix the actual values of the mixture in the pit run, as represented by the samples.

Engineers' Club of Dayton.—Announcement has been received of the opening exercises on May 14 of the newly organized Engineers' Club of Dayton, Ohio, and the dedication of the new club house. Mr. William Lodge, member of the American Society of Mechanical Engineers and president of the Lodge & Shipley Machine Tool Co., Cincinnati, was the speaker of the evening. Officers were elected and by-laws adopted.

Water Supply Problems at Santiago.—Difficulty has been found in finding a sufficient water supply for the city of Santiago de Cuba. Nearby sources have proved entirely inadequate, and engineers have not yet solved the problem. The newest proposition is to tap the River Baconao at Ramon de las Yaguas, through pipes to the city. The distance is where a dam can be built and water sent 75 miles, and the cost is estimated at $3,000,000.

The Ohio State Board of Health has ordered the city of Cambridge to build new water works by January, 1915.

Engineering and Contracting

Devoted to the Economics of Civil Engineering Design
and to Methods and Cost of Construction

Volume XLI. CHICAGO, ILL., JUNE 3, 1914. Number 22.

Village Water Supply.

In discussing the advantages of a public water supply, Mr. W. H. Dittoe, chief engineer of the Ohio State Board of Health, recently gave some very interesting figures relative to water supply conditions in the villages of the southerly half of his state. In this section, 315 villages have populations of 3,000 or under. Of the cities under 500 population 2½ per cent have public water supplies. Of the cities of 500 to 1,000 population, 1,000 to 2,000 population and 2,000 to 3,000 population, the number of cities having public water supplies are 23, 50 and 80 per cent, respectively. It is interesting to note that one village of 100 population has a public water supply. Mr. Dittoe concludes that the number of water supplies which have been installed in communities of less than 1,000 population refutes the argument that the small communities cannot afford to provide this important municipal improvement.

The latter point is well taken. There are few communities of 1,000 population, in the central and eastern portions of the country at any rate, that cannot afford a public water supply. In arguing the advantages of such a supply before the officials and taxpayers of villages, however, the health officer and engineer should do more than urge its value from the standpoint of public health and good business investment. The former consideration is now given much weight by villagers, and rightly so, but the latter point is usually entirely lost upon them and impresses them as being exceedingly far-fetched and as a promoter's "talking point."

In addition to considerations of public health considerations of convenience have great weight when they are frankly brought forth. If the people of a village, from one viewpoint, prefer to look upon a public water supply as a luxury why not let them so regard it? It is something of a luxury perhaps but one to which they are entitled and which, as a rule, they can afford. Considering the scale of living in this country and how cheerfully even our villagers pay for what they want, it becomes almost ridiculous for small communities to say that they can't afford public water supplies. Once such communities appreciate the convenience of running water in the house they are ready to undertake the installation of water works systems. More attention can well be paid to this aspect of the matter by those interested, for any reason, in promoting public water supplies.

Moving Earth With Wheeled and Fresno Scrapers.

To a dirt mover the mention of a wheeled scraper gang calls for comment. There is no element of his organization in which the average grading contractor takes a greater pride, and it has been often said, that there is no better gage of the calibre of a contractor than the operation of this outfit. Under modern conditions it is the cavalry of the grading force. When well organized and running smoothly there is no prettier sight a rough talking, hard featured crew—but it makes the dirt fly. It is the thermometer of the morale of the contractors' organization Its output reflects immediately any change in operating condition. Mules and men, dirt and weather, make a combination calling for knowledge and generalship of a high order.

The human element which enters into the organization and operation of scraper work makes it difficult to establish definite rules for determining output, but there is no study more instructive—which will lead to valuable results—than the careful analysis of the operation of such a crew. This has been done in the article in this issue of Engineering and Contracting by Richard T. Dana. In this article the experienced man will find that many of the methods and figures which he carries in his head are not only checked but the factors entering into grading work are accurately analyzed and stated.

It is undoubtedly true that many grading contractors are strongly wedded to certain methods and tools often differing materially for use on the same class of work and frequently to the exclusion of more economical methods and tools. There are a number of possible causes for this. The difficulty of becoming thoroughly acquainted with the numerous appliances available and the adaptability of each to different conditions, such as soil, labor, and motive power, is great. The human element involved in the operation of each makes a contractor leary of changing from something he has spent a good portion of his time and energy becoming thoroughly familiar with to a method or device which will in all probability requires much additional close study to operate efficiently.

It is possible also that a lack of accurate data is partly responsible for this condition. Rules for increasing the output of men are often subject to quick and radical changes; theories for machines are practically constant. The combination of the two often leads to valuable results where conditions are carefully studied and rules are not accepted as ironclad but as a guide to the economic output. They furnish also a valuable means of comparison with other work and serve as a guide in establishing methods on new work which from the start are free from many elements of chance.

It may be stated that the cost of moving dirt on one job should never be accepted literally when estimating the cost on a similar job. Costs may serve as guide boards but they are by no means equivalent to a firsthand knowledge of the road to be traveled. The mistake is often made of estimating directly from cost data. Cost data should not be used firsthand; prepare an estimate based upon local conditions and use available cost data as a check to determine whether the estimate is within the bounds of reason as judged by work already accomplished.

It must be borne in mind also when interpreting time studies, and curves and data derived therefrom, that results as shown are absolutely accurate while the work is in progress, modifying conditions due to other causes must be estimated and allowed for separately.

"Theoretical" and "Practical" Designing—The Fallacy of Attempting So to Divide Engineering Designing.

The efficient designing of structural steelwork requires the services of an engineer who is essentially practical. Unfortunately we still occasionally find an engineer who is inclined to believe that the only practical engineers are those who have gained their entire engineering knowledge from the school of experience and who have never attended an engineering college. Most engineers, however, realize that we have many practical engineers among us who are graduates of engineering schools and also many who have never received training in such schools. One of the chief advantages possessed by the engineering graduate lies in the fact that his schooling gives him a training in the fundamentals of engineering, and thus enables him to avoid many of the mistakes which the other man is apt to make while slowly acquiring his education in the school of experience.

The advisability of discussing, formally, only those things with which we are familiar, and of avoiding the formulation of definitions and rules governing subjects concerning which our knowledge is quite limited, is strikingly illustrated by a paper recently published. We should not care to give further prominence to this paper were it not for the fact that the author of it is president of an engineering company, and published his article in a well known engineering journal. As some of the statements made are absurd, and yet, because of the prominence given them, may tend to mislead men just entering into their engineering careers, we shall quote these statements and then discuss them. We have no criticism of a considerable part of the paper, which is entitled "The Practical Design of Steel Structures," where the author confines himself to statements of facts. Starting with the introduction of the article to which reference has been made we quote the following:

> Permit me to emphasize the word "practical." This qualifying word was inserted to make the subject more interesting to the general engineer. The distinction between the theoretical and the practical designer is thus described:
>
> The theoretical designing, which consists of making the preliminary plans and calculating the stresses, is generally done by men specially trained for that division of the work. These men are invariably technically educated, although some self-made men have become very proficient, in which case they have worked in conjunction with or directly under technical men. It will thus be seen that for the calculation of stresses, a college education is absolutely necessary.
>
> * * * The practical design can be subdivided into three general divisions:
>
> (1) Laying out and detailing in the office.
> (2) Fabrication in the shop.
> (3) Erection in the field.
>
> * * * The practical man generally makes the better draftsman, because what he knows he has learned by hard knocks. He goes about his work thoroughly and systematically and produces a very good picture of the members under consideration. By practical draftsman, I mean one who has had a common school education and has worked in the shop or field, preferably both, and then comes into the drawing room and learns the business from the bottom up.
>
> The technical man figures and thinks too much and fails to produce the picture which shows what is wanted. The fault with the technical man is that he has been educated to be an engineer, and when he finds that he has a certain amount of drudgery and mental work to do he gets discouraged.

From these quoted statements it will be seen that the author divides the subject of designing into two classes: "theoretical designing" and "practical designing." Such a division is absurd. One is just as "practical" as the other, and neither is "theoretical" (which means not practical or depending on theory). The making of preliminary plans most certainly should be in charge of a man who is essentially practical. The statement that "making the preliminary plans and calculating the stresses is generally done by men specially trained for that division of the work" is axiomatic—and is equally true of any kind of work which we may call to mind. It is stated that "These men are *invariably technically* educated although some self-made men have become very proficient." This statement is not only contradictory, but it also shows the author's inaccurate line of reasoning. It is evident that by the term "technically educated" the author means such men as have received their training in an engineering school. We take exception to this restricted use of the word "technical." It applies equally well to any art, or science, or even to business. The skilled painter is "technically educated," as is also the skilled mechanic, or the trained business man.

The statement that "It will thus be seen that for the calculation of stresses a college education is absolutely necessary" is also absurd. We know of many men who have never attended an engineering school, yet are very proficient in computing stresses and designing steel structures. In fact our engineering schools are now giving less emphasis to the importance of knowing how to compute stresses. They realize that this part of the work is only incidental to an engineering education, and are devoting more time to the thorough teaching of fundamental engineering principles.

Taking up the author's subdivision of what he terms "practical design" it is quite evident that "Fabrication in the shop" and "Erection in the field," two of the three classes into which he divides this subject, are not a part of design at all. It is, however, true that a knowledge of the methods used in fabricating and erecting the steelwork is essential to the efficient designer, as it will enable him so to design his structures that they can be economically fabricated and erected. In fact, we believe that much better results could be secured if the engineer who designs a structure would give more of his time to designing the erection equipment and the erection procedure.

The author's assertion that "The practical man generally makes the better draftsman, because what he knows he has learned by hard knocks" would not be open to criticism if he had not defined what he meant by a "practical" draftsman. Here it is a case of talking too much. He says "By practical draftsman, I mean one who has had a common school education and has worked in the shop or field, preferably both, and then comes into the drafting room and learns the business from the bottom up." Evidently the author believes that limited education is essential if a "practical draftsman" is to be developed. Such an argument is illogical, as education *in itself* can only help a man—it certainly is not detrimental to him. It is fortunate for the engineering graduate that most employers of engineering draftsmen do not reason as does this one. This we know to be a fact. Years ago there was prejudice against graduates of engineering schools, but the author is a "standpatter" on this proposition and is practically alone.

The statement that the "technical man" (here meaning the engineering graduate) "figures and *thinks* too much, and fails to produce the picture which shows what is wanted" is rather startling. Although engineering graduates do sometimes over-emphasize the importance of some features of college life, this is the first time to our knowledge that the charge of thinking too much has been made against them. If an engineering education *could* make every man do "real thinking" it should be made compulsory.

Evidently the author still believed that he had not made his case strong enough for he says "The *fault* with the technical man" (meaning the man with an engineering education) "is that he has been educated to be an *engineer*." This indeed is a severe indictment, yet it is made by the president of an engineering company, who himself is a member of a national engineering society. He concludes the above sentence by saying "and when he finds that he has a certain amount of drudgery and menial work to do he gets discouraged." Such a general statement is unfair to the engineering graduate, for a very large number of such men have started as draftsmen and later have risen to positions of prominence. The fact that they have been unwilling to remain draftsmen all their lives does not signify that they became discouraged, but that they possessed a worthy ambition. It is probably true that not through choice but through necessity did the man of limited engineering education continue as a draftsman until he was forced out of the organization.

We believe that it is a mistake to attempt to set apart, arbitrarily, one branch of engineering design as "theoretical," and assign to it men who have received training in our engineering schools, and another branch as "practical," and to place in charge of it men who have gained their entire knowledge of engineering from the school of experience. Either type of man may become eminently successful in any branch of engineering, provided he has the proper desire and ability to learn and the necessary application. To be successful in any branch of engineering one must be essentially practical, and this quality cannot be measured by the fact that a man has or has not received training in an engineering school. Such a training, however, we believe is advisable.

The Cincinnati Sewerage Report.

The city of Cincinnati has long been confronted with a sewerage problem. For over a quarter of a century efforts have been under way for the recognition of this problem and for its practical solution. About two years ago a constructive technical study of the broad problems of sewage collection and disposal for the entire city was begun and, based upon these studies, a report on a comprehensive plan of sewerage has just been published.

The report is of exceptional interest not only on account of its scope but from the amount of detailed information it contains. The Cincinnati sewerage engineers have had a rare opportunity to begin at the bottom and to build up a comprehensive sewerage system for a great city. A careful reading of the report convinces one that the engineers are in a fair way to make the most of their opportunity.

The report contains chapters on the history of the present systems, the underground survey, the topographic survey, new and relief sewers, intercepting sewers and creek improvements and sewage disposal. Articles based upon some of these chapters will be published in early issues of this journal in the sewerage section. The present issue contains an article on the design of new and relief sewers. Articles will follow on the underground survey, the design of intercepting sewers, and the study of the sewage disposal problem. All of these articles will be of unusual interest and value to all classes of sewerage engineers.

Decreasing the Financial Hazard Incident to the Sinking of Wells.

It has often been remarked that all contracts for pipe lines in wet materials, or in earth and rock of unknown character, are so speculative as to become, in effect, mere gambling. The natural consequences of bidding on work involving the excavation of uninvestigated materials are big losses or unfair profits. The sinking of water supply wells is a form of construction work in which the financial hazard has seldom been minimized by unit price bidding of an adequate nature such as is now frequently employed in trenching, tunneling and kindred operations in the soil. The water department of San Diego, Cal., recently received bids for sinking wells under a set of specifications which we publish in abstract in an article in the water works section of this issue.

The advantages arising both to contractor and owner from the sinking of wells on the unit price basis, here under consideration, are perhaps best indicated by reviewing present general practice in awarding well contracts and in pointing out some of the financial hazards incident to these contracts. The owner commonly specifies the diameter of well desired, fixes its location and estimates its probable depth. The contractor, without knowing the exact nature of materials to be traversed by the well, submits his bid to put down the well at a certain price per foot of depth. Frequently the price is made to vary, in the proposal, according to the final depth of the well. In proceeding under this plan the contractor must base his estimate of cost upon the materials he expects to encounter in sinking the well, and his judgment is guided by the best available logs of wells put down in the locality.

A well-drilling concern which operates extensively in any section soon collects much valuable data on underground conditions in that section. Moreover logs of wells are kept by municipal and other corporations who have had wells driven, and these records are usually available to the well contractors either at first hand or from the state geological department. Beyond the scope of these inferential data the contractor must gamble on what he will strike. It is scarcely necessary here to point out the limitations of well logs as basis for bidding on the sinking of other wells. Of course such data are valuable as far as they go but all who are at all familiar with well construction know how rapidly the formation of gathering grounds sometimes varies in short horizontal distances.

Under the San Diego specifications the unit prices submitted in the contractor's proposal are predicated upon two variables; namely, the depth and the nature of the material encountered at that depth. The contractor is therefore paid a fair price for his work, and is largely protected against the financial losses incident to striking unlooked for materials or unexpected quantities of the materials which render construction difficult. The advantage to the owner is obvious.

There are enough hazards in the well drilling business, what with the loss of tools in the well and other mishaps peculiar to this business, without gambling on the nature of the materials to be penetrated. The San Diego method of awarding well drilling contracts deserves to become widely adopted.

The International Engineering Congress at the Panama-Pacific Exposition.

There is outlined in this issue the program of papers and discussion planned for the Engineering Congress which assembles next year at the International Exposition in San Francisco. The noteworthy feature of this congress is the provision made for the wide participation of engineers of all kinds and to render available the papers and discussions. For the nominal fee of $5 any engineer may become a member of the congress and with this membership is included any one volume which may be chosen of the papers and discussions. Additional volumes can be purchased for $5 each. As prices run for books on engineering, this price of $5 per volume is low.

Another feature is worth noting. There seems to have been a systematic effort to direct the labors of the various contributors by planning in advance and in detail the subjects to be considered by each writer. Lack of co-ordination of different parts of the program of papers and want of precise definition of the field to be covered by the different contributors have been noticeable defects in the published proceedings of most similar engineering congresses. It will be a material benefit to its proceedings if the work of the Panama-Pacific Engineering Congress is directed by a carefully developed plan. To engineers who have felt that the construction of the Panama Canal should be recorded in some more available and usable form than is provided by the annual reports of the Isthmian Canal Commission, it will be a gratification to note that the canal work will be the subject of perhaps a score of co-ordinated papers prepared by the canal engineers, and planned to be collected in one volume of the proceedings of the congress.

The Canadian Government has set aside an area of 95 sq. mi. in British Columbia to be used as a national park. It is beautifully situated in the vicinity of Mount Revelstoke and will be known as Revelstoke National Park. Many natural beauties, such as glaciers, waterfalls and high mountain peaks are included in the reservation, and it is located on the line of the Canadian Pacific Railway.

Roads and Streets

The Economic Handling of Earth by Wheel and Fresno Scrapers.

Contributed by Richard T. Dana, Consulting Engineer, 15 William St., New York.

The cost of earth moving is a substantial factor in the total expense of nearly all construction work, and a considerable part of all earth moving operations is for work where it is not economical to use locomotives and cars, either because of the length of haul or because the magnitude of the work is not sufficient to justify the preparatory costs of a large plant. Under such conditions a choice must be made between wagons or carts and one or more of the various types of scraper.

It has been the experience of the writer that a majority of contractors east of the Allegheníes are unfamiliar with the relative advantages of the different kinds of scraper and do not possess the data necessary to determine for the particular conditions of their work which style and size should be most economic. A contractor who has found wheel scrapers very successful in a certain kind of earth is likely to be biased in favor of the wheel scraper for that kind of earth more or less regardless of the length of haul. Errors of judgment in a matter of this kind result in literally burying for all time money that ought to bring some benefit to somebody. Money lost by utilizing the wrong method in the field is lost to the whole community. No one gains by it, since it is spent without producing a proportionate result. It is as though a man in carrying earth in a wheelbarrow had a pig of iron fastened to the vehicle which served no purpose except to reduce the net load which the man could haul, and the net result that he could produce with his day's labor. A contractor who should discover a pig of iron screwed to the under side of his wheelbarrow would certainly unscrew it and feel that someone or something had done him a grievous wrong. To use the wrong equipment is an equally grievous wrong, but it is not always so obvious, and is far more common. The principal causes of such error are lack of data and of the formulas necessary for their application.

A careful study and analysis of scraper work was made under the direction of the writer by Mr. A. C. Haskell for the Construction Service Co. of New York, and the results are given below to enable those who have scraper work to make rapidly and conveniently those computations without which no work of this kind can economically be done. The general economic formula for transportation is as follows

Symbol.	Factor
C	The total expenses per day in cents.
w	The net load, for the average trip, in pounds, or other convenient unit
S	The speed (average) when loaded, in feet per minute
KS	The speed (average) when returning, in feet per minute.
D	The length of haul in feet.
l	The time lost in turning, resting, and wasted for an average round trip, in minutes
R	The total cost in cents per ton for transportation
W	The number of minutes in the working day.

The following facts are deducible algebraically

$\frac{D}{S}$	Time for a loaded trip, in minutes
$\frac{D}{KS}$	Time for the empty haul
$l+\frac{D}{KS}$	Actual time not occupied in transporting material, in minutes
$\frac{D}{S}\left(1+\frac{1}{K}\right)+l$	Average time for one round trip, in minutes.
$\frac{W}{\frac{D}{S}\left(1+\frac{1}{K}\right)+l}$	Average number of trips per day. This value must be an integral quantity, for the average work for any one day.
$\frac{Ww}{\frac{D}{S}\left(1+\frac{1}{K}\right)+l}$	Average total amount transported per day.
$R=C\frac{\frac{D}{S}\left(1+\frac{1}{K}\right)+l}{Ww}$	Cost of transportation per pound, or other convenient unit.

The value for C will depend upon the number of horses used in a team, whether extra teams are employed in loading, the general organization of the gang, and to a lesser extent on the cost of repairs and depreciation of the scrapers themselves. The value w of the net load will depend entirely upon the type and size of equipment, and the care with which loading is done. It is likely to vary much more with wheel scrapers than with Fresnos. In heavy ground which has not been well loosened, particularly where there are many roots of trees and occasional boulders, the wheel scrapers often fail to get more than a 60 per cent or 70 per cent load. The value for w was determined by taking the average of several hundred trips, the amount of each load being estimated by the inspector on the basis of the space occupied loose, multiplied by a density factor and afterward checked by the place measure computations. The check on the wheeler capacity was not so accurate as that with the Fresnos, in which case the computed amounts from several cellar excavations checked the estimated loads within 5 per cent.

In a well organized scraper gang the speed loaded and that light are fairly constant for constant conditions, changing as soon as the local conditions vary. These conditions are the wetness of the ground hauled over, the "sea room" available for each team and the grades and curvatures along the line of haul. The length of haul is nearly always a constantly varying quantity. Its average value was determined by measuring it at regular intervals, and averaging the figures thus obtained. The time lost in turning, loading, etc., varies somewhat but for a given set of conditions was taken as an average of a large number of observations. The data obtained in this way by an inspector who timed scrapers for two hours or so each day were afterward compared with a count kept by a boy with a tally machine at the dump, resulting in very close agreement, except that the average costs obtained by the time study method were nearly always a few per cent less than the probable actual costs kept by the tally boy over the whole day's work. The main reason for this seemed to be that when under the eye of an inspector with a watch in one hand and a note book on his knee the drivers keep their teams going a little faster when not loaded and are a little more prompt in dumping and turning than when observed by a boy.

A sample page from the inspector's note book is given by Fig. 1, showing how the notes were entered in the field, and there follows in Fig. 2 a typical report, made up at night by the inspector, of his conclusions, with the result of observations. These notes were taken by an expert observer, and would not be easy to duplicate by the ordinary timekeeper; but sufficiently good notes for maintaining excellent control over the work

Waiting.	Loading.	Transporting.	Dumping.	Transporting empty.
13:30-15:25	15:37	18:27	18:50	20:58
14:15-16:26	16:50	19:50	20:30	22:13
24:20-25:13	25:27	29:15	29:22	32:13
24:40-25:50	26:20	29:29	29:35	32:49
33:18-34:04	34:17	37:58	38:09	40:20
33:25-34:58	35:10	38:09	38:20	40:30
42:25-43:12	43:32	47:10	47:15	49:50
43:20-44:04	44:18	47:15	47:20	50:20
50:48-53:20	53:30	56:14	56:27	58:40
51:07-53:48	54:05	56:57	57:12	59:50

Footnote.—The time under each heading is that at which the performance was completed, i. e., Line No. 1 waiting at the loading point was from 13:30 to 15:25; loading from 15:25 to 15:37; transporting from 15:37 to 18:27, etc.

Observations were generally made in groups of 10 or 20 to facilitate obtaining the averages.

Times were taken upon different teams in order to get a fair average.

Fair.
Date, June 20.
Sand and some loam.
Loads fair—3 to yard, average.

Gang: 10 scrapers.
1 snatch.
1 loader.
1 dumpman.
1 foreman.
1 waterboy.

Road to dump better condition today and drivers walk.

Turn = 12 x 2.6 = 30'
D = 240 x 2.6 = 625'
D' = 225 x 2.6 = 585'

Noted on several occasions less time occupied to dump, as drivers mount scrapers without stopping them.

Fig. 1. Sample Page from Inspector's Note Book Showing Method of Keeping Notes in the Field.

can be made by a man of limited experience, once the principles are thoroughly understood. The main essential is that the observer shall be absolutely unbiased. He shall have no interest whatever in the quality or quantity of the work done, and should confine himself solely to an accurate report of the facts.

Date................ **Wheel Scrapers.**

H——— gang was hauling sand and loam from A——— Avenue, under the railroad bridge to a point close to the cement factory. The road to the dump was in very fair shape today, having dried up considerably. The drivers walked with scrapers loaded almost invariably. A decrease in time required to dump was noticed, due to some of the drivers mounting the empty scrapers while in motion, thereby cutting down the time by the amount required to stop, get on scraper and start again. Considerable time might be saved if all drivers followed this plan. Good loads were the rule, although there are many boulders, large and small, which made loading difficult at times.

Waiting at loading point. Mins.	Secs.	Loading. Secs.	Transporting loaded. Mins.	Secs.	Dumping and turning. Secs.	Transporting empty. Mins.	Secs.
1	55	12	2	50	23	2	08
2	05	30	3	00	40	1	43
0	53	14	3	48	7	2	51
1	10	30	3	09	6	3	14
0	46	13	3	41	11	2	11
1	33	12	2	59	11	2	10
0	47	20	3	38	5	2	35
0	44	14	2	57	5	3	00
2	32	10	2	44	13	2	13
2	41	17	2	52	15	2	38
Av. 1	30.6	17.2	3	09.8	13.6	2	28.3

D .... 625'
D' .... 585'
S .... 197' /min.
KS .... 236 min.
K 1.2
l .... 2 m. 01.4 s.
D S D'KS...... 5' 38.1"

W .... 0.33 c. y. = 835 lbs.
C .... 739.5 cts.

10 scrapers	$55.00
1 snatch	7.50
1 loader	2.25
1 dumpman	1.65
1 foreman	6.55
1 waterboy	1.00
10 C =	$73.95

$$R = \frac{739.5}{600} \times \frac{7.659}{0.33} = 28.6 \text{ cts. per c. y.} = 4.58 \text{ cts. per c. y. per 100' hau} = 0.0113 \text{ cts per lb.}$$

Fig. 2. Typical Report from Field Notes.

In the following tables the columns headed "Transportation Cost" show the cost for actually moving the material, while the columns entitled "Static Cost" are for the cost of waiting at the loading point, loading, dumping and miscellaneous delays. No account is taken of extraordinary delays, such as breakdowns, etc.

WHEEL SCRAPER WORK.

The following observations of wheel scrapers cover a period of some three months and were made under various conditions.

The wheel scraper is a modified form of the drag scraper, being suspended between two wheels in such manner that the pan drags along the ground when loading and then, when full, may be raised by a system of levers, so that it rides suspended several inches above the ground. The operation of loading ordinarily requires the assistance of a two or three-horse snatch team and driver, and a "loader" whose duty it is to lower the pan, guide it while being filled and to raise it when full. At the dumping point the pan is unhooked and lowered and the dumpman by an upward thrust upon the attached handle causes the pan to make a quarter turn thus dumping and partially spreading the material.

Distinction has been made between wheelers in loam, clayey loam (Table I), etc., and wheelers in sand, fine gravel, etc., owing to the fact that the nature of the soil materially affects the size of load, and consequently the speed of transportation more or less. Although it may take less time to get a full load in soft sand, it usually takes more time to get started after the snatch team has been unhooked owing to the wheels being deeply buried. And during transit the sand will waste much more than the loam, so that the load actually dumped is considerably less.

In Table I column 12 gives values of R in the transportation formula. Table II gives costs on the same kind of work at about the same time, taken by an ordinary timekeeper without time study and is inserted for comparison.

From Table I it will be seen that the average cost per cubic yard was 6.2 cts. for the fixed or static cost, which includes cost of loading, dumping and idle time, and 4.1 cts. per 100 ft. of haul for the transportation cost, based on the time study figures.

TABLE 1.—WHEEL SCRAPERS IN LOAM AND CLAYEY LOAM.
(Arranged in order of static charge.)

	No. Sc.	D.	D'.	S.	KS.	K.	Load C. Y.	l Min.	Loading time, secs. (included in static cost).	Other "l" time, sec.	Total cost. Cts. per cu. yd.	Static cost, cts.	Transp. cost. Cts. cu. yd.	Transp. cost per 100 ft. haul. Cts. cu. yd.
	6...	385	445	212	233	1.06	0.38	1.10	26	40	15.2	3.9	11.3	3.0
	5...	190	225	208	225	1.08	0.37	1.23	28	46	11.0	4.3	6.7	3.5
	5...	525	525	194	232	1.20	0.33	1.03	30	32	23.2	4.3	18.9	3.6
	5...	345	345	218	218	1.00	0.31	1.00	30	39	16.5	4.5	12.0	3.5
	5...	235	285	142	197	1.38	0.41	1.24	31	43.5	14.5	4.7	9.8	4.2
	7...	530	530	182	230	1.26	0.37	1.41	26.2	58.5	23.6	5.0	18.6	3.5
	5...	350	350	206	232	1.13	0.33	1.26	32	43.5	16.7	5.3	11.4	3.3
	7...	570	570	222	260	1.17	0.37	1.48	25.3	63.3	23.0	5.5	17.5	3.1
	4...	350	350	226	312	1.38	0.33	1.10	34.2	32	18.5	5.5	12.0	3.4
	5...	330	400	222	238	1.07	0.39	1.58	39	56	18.6	6.2	12.4	3.8
	7...	325	520	206	255	1.24	0.30	1.54	19.3	73.2	22.8	6.8	16.0	4.9
	4...	125	190	268	150	0.56	0.42	2.05	34	89.0	15.2	8.3	6.9	5.6
	6...	275	625	216	244	1.13	0.30	1.89	21.2	92.4	27.0	8.9	18.1	6.6
	4...	195	195	197	240	1.22	0.35	1.90	36.8	77.1	18.0	9.2	8.8	4.6
	7...	250	380	218	216	0.99	0.30	2.23	17.9	115.9	23.0	10.0	13.0	6.2
Min.	4...	125	190	142	150	0.56	0.30	1.00	17.9	32.0	11.0	3.9	6.2	3.0
Av.	5.5..	332	395	209	232	1.11	0.35	1.54	28.7	60.1	19.1	6.2	12.9	4.1
Max.	7...	570	625	268	312	1.38	0.42	2.23	39.0	115.9	27.0	10.0	18.9	6.6

In the above table abbreviations are as follows:

No. Sc.................Number of scrapers at work in gang.
D ....Length of loaded haul, feet.
D' ....Length of empty haul, feet.
S ....Rate of travel, loaded, feet per minute.
KS ....Rate of travel, empty, feet per minute.
K ....Ratio of empty speed to loaded speed.
Load C. Y..............Size of load carried in cubic yards, place measure.
l ....All time except that in actual transit to or from dump.
Other "l" time........Other lost or idle time; that in addition to loading time.

The observations in Table III were made where a road had been cleared through the woods. The soil, a rich loam, was more or less interspersed with roots, to extricate which required the labor of two men. In this instance the static cost is high, due in a measure to the time required to load. Owing to the roots running through the soil sometimes it was necessary to make more than one attempt to load, but good full loads were obtained, which is really more essential than saving a few seconds and starting for the dump with a half loaded scraper.

The principal thing to be called to attention here, however, is a fault very frequently observed, the wrong apportionment of scrapers to length of haul. In this case there were too many for so short a haul, thereby causing frequent delays both at the loading point where they became bunched and en route to the dump, which forced up the static cost. The rate of progress in transit was also reduced, especially on the return from the dump, as is shown by *KS*, when the teams were obliged to slow down to allow those ahead room to turn, load, etc. This is a state of affairs which should never be permitted; scrapers should maintain a steady progress in both directions, and at no time should a scraper be obliged to wait idle at the loading point for the ones ahead to be loaded.

The observations in Table IV were taken where the top soil was very dry, clouds of dust being raised at each attempt to fill a scraper.

Both the static cost and the transportation cost per 100 ft. of haul are excessive, the former is due partially to the time wasted at the dump, which observation has shown to be prolonged beyond all necessity. Drivers are wont to stop, take long turns about, etc., when really there is no need of stopping at all, and the scraper should be turned about immediately after dumping, in the shortest possible radius. The excessive transportation cost per 100 ft. haul, to which may be directly traced the high cost per cubic yard, was owing to the extremely long empty haul D^1 as compared to the loaded haul *D*, whereby considerable time was consumed. The haul should always be the shortest route to the dump, and the return, if it is not feasible to use the same route, should be as direct as possible. The loads were small, due to the dry, dusty nature of the material, which could not be heaped, and wasted considerably en route to the dump, which, of course, runs up both static and transportation costs.

The observations of Table V were made where a gang was removing top soil from a piece covered with tall weeds, the roots of which prevented very full loads. The soil was dry and dusty.

Here is another example of too many scrapers for the length of haul. This fact, combined with the small loads, produced the high costs. The time "Waiting and Preparing to Load" is altogether too high. The speed *KS* on the return is less than *S* loaded speed. When this state of affairs exists it can invariably be traced to too many scrapers, for they become bunched about the loading point and the returning drivers, seeing this, slow down, thus wasting considerable time.

In the formula, $R = C \cdot \dfrac{D/S(1+1/k)+l}{W \cdot W}$

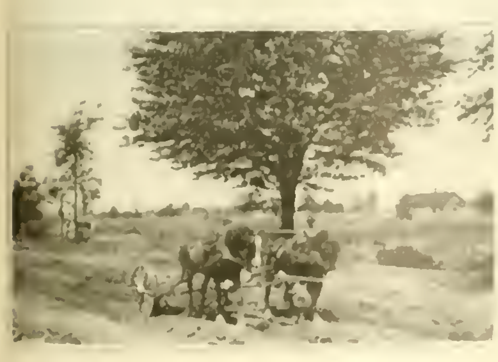


Fig. 4. Device Permitting Lateral Play of Pole in Wheel Scraper Work.

all the quantities may have fixed values determined from observation assigned to them, except *W* and *D*, the load and haul, which will vary according to the kind of scraper employed and the nature of the work

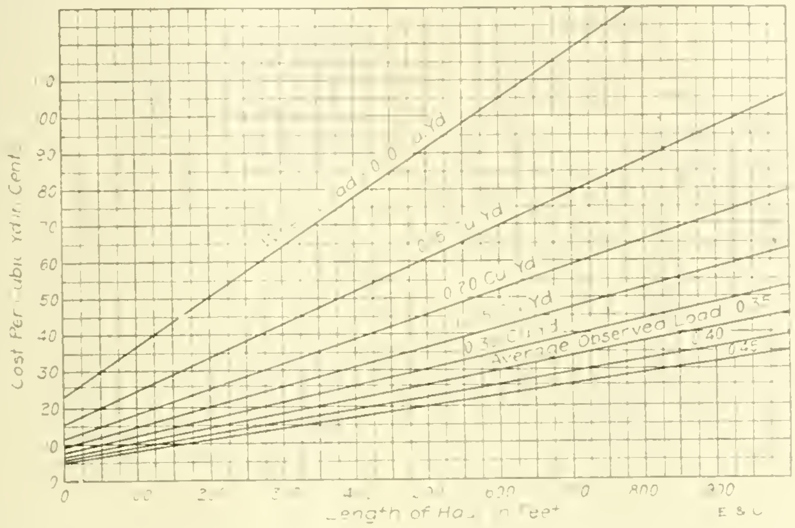


Fig. 3. Curves Showing Costs Per Cubic Yard of Handling Loam and Loam Clay with Wheel Scrapers for Various Sizes of Load and Length of Haul.

DATA USED IN CALCULATING CURVES FOR WHEEL SCRAPERS IN LOAM AND CLAYEY LOAM.

C	896 cts.
w	Varies.
l	1.54 min.
W	600 min.
D	Varies.
S	209 ft. min.
K	1.1.
1/K	0.91

Value for C for 5-scraper gang determined as follows:

10 mules at $1.75	$17.50
5 drivers at $1.95	9.75
1 loader at $2.25	2.25
1 dumpman at $1.75	1.75
1 3-mule snatch at $5.25	5.25
1 3-mule snatch driver at $2.25	2.25
1 D. R. at 3 per cent per month	1.07
Foreman at $4.00	4.00
Waterboy at $1.00	1.00
Total	$44.82
Per scraper	8.96

Value of scraper	$48.00
Value of harness	50.00

$$R = \frac{1}{w}\left[\frac{C.\,D.\,(1+1/k)}{S\,W} + \frac{l\,C}{W}\right] = \frac{1}{w}\left[\frac{896\times 1.91\,D}{209\times 600} + \frac{1.54\times 896}{600}\right] = \frac{1}{w}\left[0.01365\,D + 2.30\right]$$

	Costs.				Costs.		
w.	D = 100'.	500'.	1000'.	w.	D = 100'.	500'.	1000'.
.10	36.65	91.25	......	.30	12.22	30.42	53.17
.15	24.43	60.83	106.33	.35	10.47	26.07	45.57
.20	18.32	45.62	79.75	.40	9.16	22.81	39.87
.25	14.66	36.50	63.80	.45	8.14	20.28	35.44

Curves, Fig. 3, may be drawn showing costs for varying loads and lengths of haul.

When operating in sand and gravel wheel scrapers are subject to less difficulty in loading, and the material, although more evenly loaded, is likely to dribble out in transit. Table VI gives the results of a large number of observations under these conditions.

The observations of Table VII were made where the material was a mixture of sand and loam, without enough of the latter, however, to lend stability to the whole. Boulders were numerous, often interfering with the obtaining of good loads.

The above costs are a good example of how conditions affect results, and the above method of analysis readily shows this. The static cost, 8.4 cts. per cubic yard, is somewhat above the average, due partly to the boulders making loading difficult, and partly to the small loads carried to the dump. But it is the high transportation cost which attracts attention. This was due to the road to the dump being exceedingly rough and muddy after recent rains, the scrapers sinking at times almost to the hubs. The loads were consequently small, wasting away, and the rate of moving (*S* and *KS*) materially reduced. Observations were taken in the afternoon of the same day when a new and less difficult path was used, and the effect was shown directly, S increasing to 191 ft. per minute and *KS* to 231 ft. per minute, and the transportation cost being reduced 1.1 cts. per cubic yard.

The observations of Table VIII were made where the material handled was a coarse sand with a sprinkling of loam.

In this instance the reverse of the last case cited is true. Here the transportation cost is normal, but the static cost (which shows fluctuations in cost due to all operations except actual transportation), is very high. Small loads were again responsible in part, but the chief reason was because of a long turn of about 220 ft. around a pool of water at the dump. The time consumed in making this extra long turn after the material had been dumped so increased *l*, the time when the scrapers are not actually engaged in the handling of material, that in consequence the static cost was excessive. It will be noticed in the time study that two or three of the scrapers turned about shortly without waste of time, and the foreman should have seen that all did likewise.

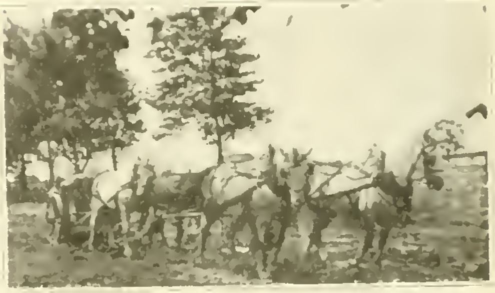


Fig. 5. Typical Arrangement of Teams in Wheel Scraper Work.

TABLE II.—WHEEL SCRAPERS IN LOAM AND CLAYEY LOAM.

No. of scrapers.	D.	Load. cu. yds.	Total cost per cu. yd.	Total cost per 100' haul.
4....	305	0.33	25.6*	8.4*
4....	130	0.33	24.6	15.9
5....	180	0.33	18.0	10.0
5....	300	0.34	22.8	7.6
4....	235	0.33	23.2	9.8
5....	160	0.37	16.7	10.4
4....	225	0.33	23.4	10.4
4....	200	0.33	20.6	10.3
4....	225	0.33	21.4	9.5
5....	180	0.33	23.0	12.8
4....	550	0.33	18.9	3.4
4....	290	0.33	18.0	6.2
5....	290	0.33	19.6	6.8
4....	300	0.33	22.8	7.6
5....	300	0.29	23.0	7.7
5....	250	0.31	19.4	7.8
4 & 5....	262	0.30	21.0	8.1
4....	180	0.29	15.1	8.4
4....	260	0.33	22.0	8.5
4....	325	0.33	24.0	7.4
4....	350	0.33	26.2	7.5
4....	250	0.33	23.7	9.5
Min. 4....	130	0.29	15.1	3.4
Av. 4.4....	261	0.33	21.5	9.0
Max. 5....	550	0.37	26.2	18.9

*Cents.

The observations of Table IX were made where the material was a coarse dry sand which wasted a great deal en route to the dump.

Observation has shown that high costs are more often due to carrying poor loads to the dump than to any other cause. It will be noticed that in the above gang a shovelman was employed whose duty it was to follow a scraper as it was being loaded and without delaying the progress of the work to fill up the scraper as much as possible. By this means the wedge-shaped space at the back of the scraper, invariably left empty when using a snatch team alone, was filled. Besides causing much larger loads to be delivered at the dump, the scheme prevented the drivers from standing on this space and riding the scrapers loaded, a thing frequently observed under the excuse that it was necessary to keep the scrapers balanced and prevent the material wasting out the front, but which should under

Fig. 6. Wheel Scraper Just After Being Dumped.

TABLE III.—WHEEL SCRAPER WORK, OBSTRUCTED BY ROOTS.

	Loading.		Transporting loaded.		Transporting empty.		Delays, including dumping.	
	Min.	Sec.	Min.	Sec.	Min.	Sec.	Min.	Sec.
	0	38	0	20	1	22	1	00
	0	27	0	18	0	57	2	18
	0	37	0	31	1	30	1	47
	0	38	0	23	1	06	1	11
	0	33	0	20	0	59	1	41
	0	27	0	31	1	23	1	33
	0	30	0	21	1	20	1	15
	0	45	0	53	1	24	2	01
	0	39	0	35	1	28	1	14
	0	34	0	27	1	10	1	04
Av.	0	34	0	28	1	16	1	29

D—Loaded haul.......125′
D′—Empty haul.......190′
S—125′ 28 sec.........268 ft. per min.
KS—190′ 1 m., 16 s....150 ft. per min.
K—150 268 ....0.56
t ....2 min. 03 sec.
D/S + D′/KS.........1 min. 55 sec.
w ....0.425 cu. yds.
C ....40.95/4 = 1024 cts.
W ....600 min.

4 scrapers at $5.50	$22.00
1 snatch team and driver	7.50
1 loader	2.25
1 dumpman	1.65
1 waterboy	1.00
1 foreman	6.55
	$40.95

$$R = \frac{1024}{600} \cdot \frac{3.77}{0.425} = 15.2 \text{ cts. per cu. yd.} = 8.2 \text{ cts. static} + 5.5 \text{ cts. per 100-ft. haul}$$

no conditions be allowed. The material here was very soft, however, and a great deal wasted off, as the haul was rather long and rough Front gates upon the scrapers would have made the retaining of the good loads produced by the shovelman's efforts possible, and correspondingly decreased the unit costs.

In order to plat curves for various loads and length of haul for wheel scrapers in sand, average values would be substituted in the formulas as follows, on the basis of a 5-scraper gang:

C ....896 cts.
I ....1.75 min.
w ....variable.
S ....215 ft. per min.
K ....1.1.
1/K ....0.91.
D ....variable.
W ....600 min.

$$R = \frac{1}{w}\left(\frac{896 \times 1.91\,D}{215 \times 600} + \frac{896 \times 1.75}{600}\right) = \frac{1}{w}\,(0.01327\,D + 2.61)$$

w.	D = 100′.	500′.	1000′.
.10	39.37	.....	.....
.15	26.25	61.63	.....
.20	19.68	46.22	.....
.25	15.75	36.98	63.52
.31	12.70	29.82	51.23
.35	11.25	26.41	45.37
.40	9.84	23.11	39.70
.45	8.75	20.54	35.29

The time studies given above show how important it is to have full loads and the proper apportionment of the number of scrapers to the length of haul. In order to make this latter point, which is of prime importance, more specific, one or two illustrations will be given to show: (1) How, if too few scrapers are at work upon a given haul, the addition of another will lessen the cost per cubic yard, and (2) if there is already a sufficient number at work to produce economical results, how the addition of one more may make the unit cost per cubic yard higher, and in addition may actually reduce the total amount of material handled.

The following observations were made where the material handled was a loam and clay mixture, rather sticky, but which loaded well and did not waste on the way to the dump. The length of haul was about 325 ft From 6:45 a. m. to 2 p. m., 6¼ hours, omitting one hour at noon, with four scrapers working 330 loads were dumped = 110 cu. yds. at three loads to a yard.

4 scrapers at $5.50	$22.0
1 snatch team and driver	7.5
1 loader	2.2
1 dumpman	1.6
⅓ foreman	2.1
⅓ waterboy	0.3
	$35.91

$$\text{Cost per cu. yd.} = \frac{6¼}{10} \times \frac{3591}{110} = 20.4 \text{ cts}$$

per cu. yd. = 6.3 cts. per cu. yd. per 100-ft haul.

$$\text{At the above rate of handling } \frac{10}{6¼} \times 330 =$$

528 loads, or 176 cu. yds. would be dumped per 10-hour day.

From 2 p. m. to 5:45 p. m., 3¾ hours, with five scrapers working, 252 loads were dumped = 84 cu. yds.

Cost: As above, with addition of 1 scraper = 41.41.

$$\text{Cost per cu. yd.} = \frac{3¾}{10} \times \frac{4141}{84} = 18.5 \text{ cts.}$$

per cu. yd = 5.7 cts. per cu. yd. per 100-ft. haul.

$$\text{At above rate of handling } \frac{10}{3¾} \times 252 = 672$$

loads or 224 cu. yds. would be dumped per 10-hour day.

Thus it will be observed that the addition of a scraper in this instance was all for the better, as for an increase of 15 per cent in cost, 27 per cent more material would be handled, per day at a decreased cost of 10 per cent per cubic yard and 10.5 per cent per cubic yard per 100-ft. haul.

Another instance was where a coarse sand was hauled about 325 ft. The loads were not very good and the costs were rather high, but the case serves well as an example of the point in view. From 6:45 a. m. to 12 m., 5¼ hours, with five scrapers working, 275 loads were dumped = 75 cu. yds. The cost was:

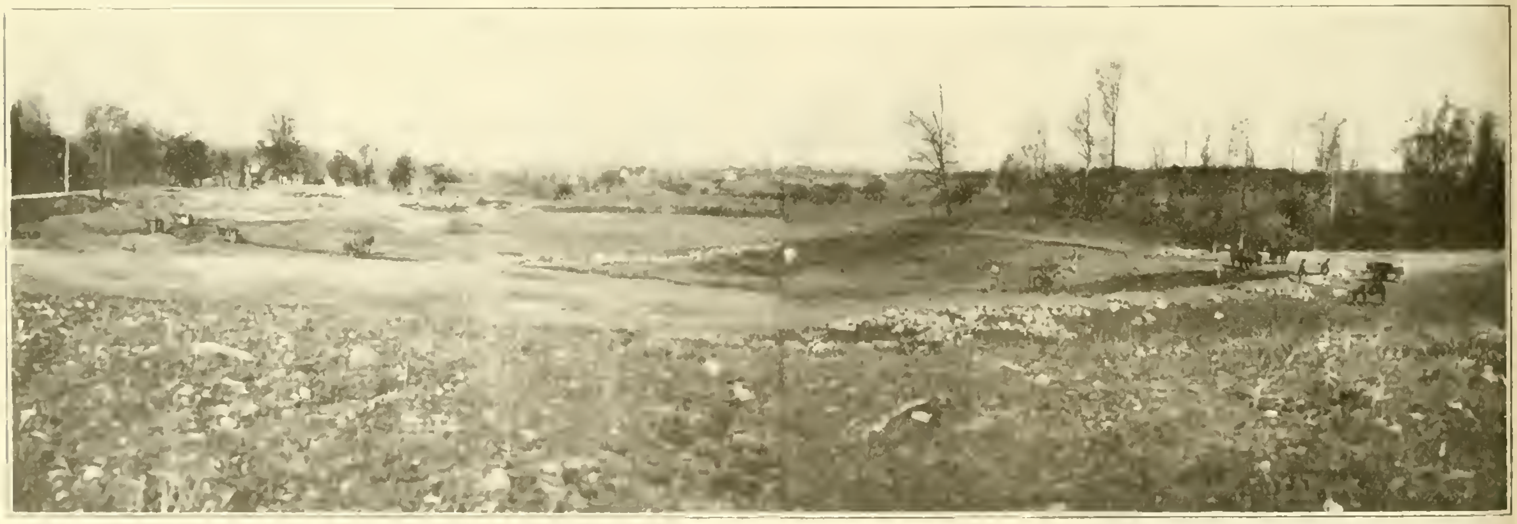


Fig. 7. Wheel Scraper Work on a Haul of 1,040 Ft. Which Was Not Economical.

TABLE IV. WHEEL SCRAPER WORK IN VERY DRY LOAM.

	Waiting and preparing to load.		Loading.		Transporting loaded.		Dumping.		Transporting empty.		Delays.	
	Min	Sec.	Min.	Sec.	Min.	Sec.	Min.	Sec.	Min	Sec.	Min.	Sec.
	0	33	0	31	1	26	0	17	2	31	0	25
	0	35	0	20	1	03	1	10	2	05	0	35
	0	13	0	23	1	24	0	20	2	38	.	..
	0	44	0	14	1	21	0	26	2	49	0	25
	0	44	0	18	1	07	0	15	2	14	0	30
Av.	0	33.8	0	21.2	1	16.2	0	29.6	2	33.4	0	23

D—Loaded haul ....275′
D′—Empty haul.............625′
S—275 1 m. 16.2 s....................216′ min.
KS—625 2 m. 33.4 s..................244 min.
K—244 216 ....1.13
I ....1 min. 53.6 sec
D/S + D′/KS ....3 min. 49.6 sec.
w ....0.296 cu. yds.
C—5030/6 ....838.33 cts.
W ....600 min.

Cost of 6 scraper gang .$50.30

$$R = \frac{838.33}{600} \cdot \frac{5.72}{0.296} = \text{[illegible] cts. per cu. yd.} = 8.3 \text{ cts. static cost} + 6.6 \text{ cts. per 100′ haul.}$$

5 wheelers at $5.50	$27.50
1 snatch team and driver	7.50
1 loader	2.25
1 dumpman	1.65
⅓ foreman	2.18
⅓ waterboy	0.33
Total	$41.41

$$\text{Cost per cu. yd.} = \frac{5¼}{10} \times \frac{4141}{75} = 29 \text{ cts.}$$

= 8.9 cts. per cu. yd. per 100-ft. haul.

At the above rate of handling, 525 loads, or 143 cu. yds. would be dumped per 10-hour day.

From 1 p. m. to 5:45 p. m., 4¾ hours, with ix scrapers working, 324 loads = 88 cu. yds. ere dumped.

Cost: As above plus one wheeler = $16.91.

Cost per cu. yd. $= \frac{4\frac{3}{4}}{10} \times \frac{4691}{88} = 25.3$ cts. $= 7.8$ cts. per cu. yd. per 100-ft. haul.

At this rate of handling 683 loads, or 185 cu. yds., would be dumped per 10-hour day. Thus for an increase in cost of 13 per cent, 29 per cent more material would be handled per day, at a decreased unit cost of 18½ per cent per cubic yard, and 14 per cent per cubic yard per 100-ft. haul.

It is not easy to comprehend at first glance the effects produced by having too many scrapers for a certain haul. It seems natural, without analysis, to suppose that the more scrapers there are working, the more work will be accomplished. Proof of this was evidenced by such conditions existing. In order to look more closely into the details of this case, it is worth while to present the complete time study, Table X. The material was a mixture of clay and sand, with enough of the latter to cause it to break up well when plowed. Seven scrapers were at work.

At the above rate of hauling 210 cu. yds. of material would be dumped per 10-hour day.

The observations in Table XI were made when eight scrapers were working in the same material and hauling to the same distance.

At the above rate of handling $\frac{600}{7.03} \times 8 \times 0.3$ 205 cu. yds. would be dumped per day.

Thus, be it observed, actually less material was handled with eight scrapers than with seven, so that in consequence the unit costs per cubic yard were higher. This result was due to the fact that the time lost at the loading point was nearly doubled (compare time studies) owing to the scrapers getting bunched up there and being obliged to await their turn to be filled with the snatch team; the time of transportation was materially increased owing to the enforced reduction of the rate of traveling due to overcrowding, so that the time necessarily lost by the gang due to this congestion more than overbalanced the extra amount of material which an additional scraper would ordinarily handle.

The above presents a typical example of conditions very frequently observed, conditions

Fig. 9. Loading Fresno Scraper.

for the existence of which there is no excuse, especially when the work is being carried on under the supervision of a man of long experience in such work, who, if he opened his eyes to facts and took an interest in his work, could [illegible] but determine what constitutes the efficient handling of his gang. But experience has proved that the great majority of foremen do not take enough interest to sit down after working hours and figure out how

TABLE VI - WHEEL SCRAPERS IN SAND.

Sc. No.	D.	D'.	S.	KS.	K.	Load, cu. yds.	1 min.	Loading time, Seconds.	Other "l" time, Seconds.	Cost per cu. yd. Cts.	Static cost, Cents.	Transp. cost Cents.	Transp. cost per 100 ft. of haul, cts.
5...	425	520	220	193	0.875	0.426	1.05	22.5	40.5	19.7	3.7	16.0	3.8
6...	500	625	246	244	0.99	0.426	1.14	25.6	42.7	20.0	3.9	16.1	3.2
6...	490	607	256	258	1.00	0.408	1.21	24.0	48.5	19.1	4.3	14.8	3.0
10 ..	1020	1020	216	247	1.14	0.296	1.30	19.6	58.8	42.4	5.4	37.0	3.6
7...	600	650	248	272	1.10	0.300	1.22	12.1	61.3	27.4	5.5	21.9	3.7
9...	1040	1040	224	276	1.23	0.296	1.31	18.5	60.2	42.6	5.7	36.9	3.5
9...	690	665	218	242	1.11	0.333	1.54	16.3	76.1	28.2	5.8	22.4	3.2
9...	845	835	216	234	1.08	0.296	1.50	17.1	72.8	38.0	6.4	31.6	3.7
8...	690	690	218	230	1.05	0.296	1.54	17.5	75.1	35.0	7.0	28.0	4.1
10...	760	800	218	230	1.06	0.250	1.46	18.8	65.0	41.5	7.2	34.3	4.5
9...	880	920	187	226	1.21	0.300	1.75	18.4	86.5	44.5	7.4	37.1	4.2
10...	625	585	197	236	1.20	0.330	2.02	17.2	104.2	28.6	7.6	21.0	3.4
9...	855	860	224	225	1.00	0.330	2.14	23.3	105.2	36.8	8.1	28.7	3.4
9...	495	545	204	234	1.15	0.260	1.66	15.1	84.0	31.2	8.1	23.1	4.7
8...	600	650	226	262	1.16	0.300	1.91	16.5	98.2	30.7	8.3	22.4	3.7
10...	825	825	205	235	1.15	0.296	2.02	19.8	101.4	39.8	8.4	31.4	3.8
10...	585	625	181	212	1.17	0.220	1.50	18.6	71.4	43.0	8.4	34.6	5.9
8...	650	650	216	236	1.09	0.296	1.90	20.7	93.2	34.8	8.6	26.2	4.0
9...	655	575	199	234	1.17	0.230	1.57	15.7	78.5	40.2	8.6	31.6	4.8
9..	535	575	210	231	1.10	0.333	2.32	19.4	120.1	28.0	8.8	19.2	3.6
10...	960	960	217	243	1.12	0.296	2.10	22.7	103.2	43.5	8.7	34.8	3.6
9...	625	585	198	221	1.13	0.330	2.60	18.1	137.7	32.0	10.0	22.6	3.5
10...	875	875	199	226	1.14	0.296	2.47	22.9	125.6	44.8	10.3	34.5	3.9
9...	585	610	220	259	1.18	0.260	2.89	16.7	156.8	38.5	14.1	24.4	4.2
Min.5...	425	520	181	193	0.87	0.220	1.05	12.1	40.5	19.1	3.7	14.8	3.0
Av. 8.7.	700	720	215	238	1.11	0.309	1.75	19.1	86.3	31.6	7.5	27.1	3.9
Max.10..	1040	1040	256	276	1.23	0.426	2.89	25.6	156.8	44.8	14.1	37.1	5.9

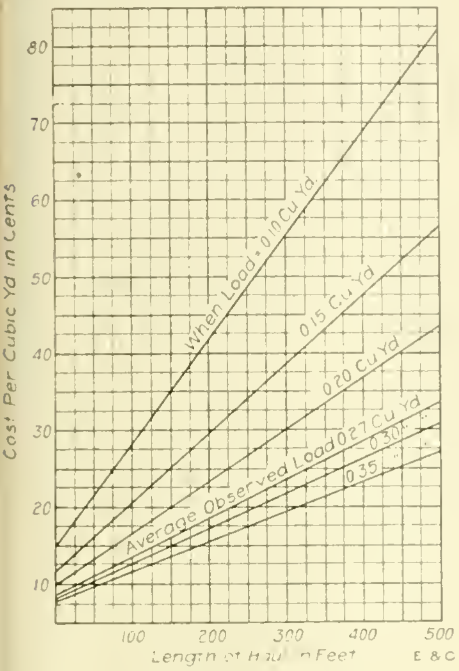


Fig. 8. Curves Showing Cost Per Cubic Yard of Handling Loam, Sand, Etc., With Fresno Scrapers for Various Sizes of Load and Length of Haul.

Five scrapers in standard gang. Three animals per scraper.

Mules at $1.75	$26.25
Drivers at $1.95	9.75
Loader at $1.95	1.95
I. D. R. at 3 per cent per month	0.98
Fireman at $1.00	4.00
Waterboy at $1.00	1.00
Total	$43.93
Per scraper C	8.79

Value of scraper	$13.75
Value of harness	50.00

$$R = \frac{1}{w}\left[\frac{879 \times 1.95\ D}{212 \times 600} + \frac{879 \times 0.67}{600}\right] = \frac{1}{w}\left[0.0135\ D + 0.98\right] = \begin{cases} \frac{1}{w}(2.33) \text{ for } 100'. \\ \frac{1}{w}(3.68) \text{ for } 200'. \\ \frac{1}{w}(5.03) \text{ for } 300'. \\ \frac{1}{w}(6.38) \text{ for } 400'. \\ \frac{1}{w}(7.73) \text{ for } 500'. \end{cases}$$

w	D = 100'.	200'	300'	400'	500'.
.10	23.3	36.8	50.3	63.8	77.3
.15	15.5	24.5	33.5	42.5	51.5
.20	11.6	18.4	25.1	31.9	38.6
.27	8.6	13.6	18.6	23.6	28.6
.30	7.8	12.3	16.8	21.3	25.8
.35	6.7	10.5	14.4	18.2	22.1

For 5 scrapers.

TABLE V - WHEEL SCRAPER WORK IN DRY SOIL FULL OF TALL WEEDS

	Waiting and preparing to load.		Loading.		Transporting loaded		Dumping		Transporting empty		Delays	
	Min.	Sec.	Min.	Sec.	Min.	Sec.	Min.	Sec.	Min.	Sec.	Min.	Sec.
	1	18	0	19	1	10	0	35	1	[illegible]		
	2	08	0	17	1	26	0	26	1	56		
	1	25	0	16	1	20	0	14	2	0	0	10
	0	11	0	14	1	00	0	11	1	18	0	59
	0	29	0	11	1	01	0	31	1	22		
	1	52	0	16	1	10	0	27	1	52	0	13
	1	28	0	22	1	01	0	39	1	45		
	1	05	0	24	0	54	0	33	1	36		
	1	38	0	23	0	59	0	18	1	37		
	2	10	0	16	1	[illegible]	0	13	1	52		
Av	1	22.4	0	17.9	1	08.[illegible]	0	[illegible]	1	[illegible]	[illegible]	8.2

D .... 250'
D' .... 280'
S = 250, 1 min., 08.5 sec. [illegible] per min.
KS = 280, 1 min., 15.3 sec. [illegible] per min.
K = 216 218 .... 0.99
t .... [illegible] min. [illegible] sec.
D S + D' KS .... 2 min. [illegible] sec.
w .... 0.296 cu. yds.
C = 5.80 7 .... 797 cts.
W ....

Cost of 7 scraper gang $[illegible]

$R = \frac{797}{600} \times \frac{5.13}{0.296} = 23.0$ cts. per cu. yd. [illegible] cts. static [illegible] cts. per 100 ft. of haul the efficiency of their gang may be increased during working hours. Scraper work may be divided into the following: Loading, transporting and dumping. Each of these operations is distinct in itself and yet each so depends upon the others that if there is any flaw in one the whole work must show the effect. For example, a few moments' observation of a piece of work shows that the scrapers are constantly in motion with no idle time at the loading point except that required to turn and get into position for loading and the progress to and from the dump is brisk, but there are periods when the snatch team is idle for several minutes waiting for a scraper to load and the dumpman sits idly upon his shovel.

Fig. 10. Loaded Fresno Scraper on Way to Dump.

Again, if the loading team is hooking into one scraper after another in rapid succession and the dumpman has no sooner spread one scraper load than the next team comes along, but gathered around the loading point idly awaiting their turn to be loaded are several scrapers, the drivers laughing and joking, the horses nosing about on the ground, the progress to the dump reduced because of congestion, is it not self-evident that too many scrapers are on the work and enough should be taken off to keep them moving all the time? When things are going briskly the entire atmosphere of the work is charged with energy, felt alike by man and beast, but when obliged to wait for minutes at a time in idleness a feeling of inertia pervades the work which, in its results, it must necessarily reflect. Keep everything in motion. See that the loader, snatch team, and dumpman are kept busy, that the scrapers are continually on the move, and work will be performed economically. Have regular intervals for letting the whole gang rest, if necessary, but while at work let each individual unit of the whole be performing its function to the best of its capacity.

The greatest trouble with wheel scrapers is likely to be caused by the heavy pressure on the collars of horses and mules, due to loading in heavy ground. For this reason the collars should be made to fit as perfectly as possible, and the animals examined every night for sores on the necks or withers.

When loading wheelers with a snatch team the chain should be hooked to a point as near as possible to the scraper itself, otherwise the pull of the snatch team will throw a heavy load on the backs of the wheeler team.

An excellent device observed in the suburbs of Cleveland, O., is shown in Fig. 4. By this means the pole has a very considerable lateral play, very much to the relief of the wheel team when strain comes upon the snatch.

Figure 5 illustrates a typical arrangement of teams, removing earth in street grading.

Figure 6 shows a wheeler just after having been dumped, two men doing this, unassisted by the driver. Only one extra man is necessary, if helped by the driver. These illustrations are from photographs taken on a well organized and highly efficient piece of work. Figure 7, composed of two joined photographs, shows the general conditions on a haul of 1,040 ft., which was not economical. No amount of good management in the gang will overcome the handicap of a haul so long as to require an altogether different kind of equipment. The cost of the work shown in Fig. 7 for the transportation alone was 36.9 cts. per cubic yard.

The working speed of a team is such that when heavily loaded the team proceeds a little more slowly than the ordinary man will walk, but when "light," horses (more than mules) are apt to move faster than the ordinary walking gait of the driver. Consequently it is a good plan to require all drivers to ride in the scraper (which can be done with wheelers, but not with Fresnos) when light, and never to sit on the loaded scraper.

When dumping a wheel scraper it is important, so far as possible, to have the team on ground at least as high as that on which the scraper rests. Otherwise the heavy load that always comes on the collar or saddles by the act of dumping will be increased by gravity, often to the great distress of the animals.

In loading a scraper, care should be exercised not to allow the team to pull it farther and overload it after it is once full, since such extra effort is utterly useless and is very exhausting. Conversely, each load should be a full one. The first mark of a badly handled job is improper and variable loading of the vehicles.

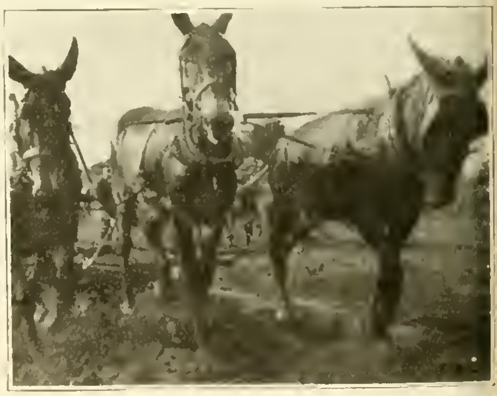


Fig. 11. Harness Arrangement in Fresno Scraper Work.

It should be borne constantly in mind when laying out and directing earth moving work with small equipment that the "normal haul" for a wheel scraper is longer than for a fresno and shorter than for a one-horse cart.

FRESNO SCRAPER WORK.

With this type of equipment, each scraper is hauled by from two to four horses, or mules, depending upon its capacity; but the services of a snatch team and driver and usually of a

TABLE VII.—WHEEL SCRAPER WORK IN SANDY LOAM.

	Waiting and preparing to load.		Loading.		Transporting loaded.		Dumping and turning.		Transporting empty.	
	Min.	Sec.	Min.	Sec.	Min.	Sec.	Min.	Sec.	Min.	Sec.
	0	45	0	28	3	37	0	15	2	50
	1	02	0	15	3	03	0	15	3	02
	0	40	0	16	3	12	0	12	3	10
	1	10	0	14	3	16	0	33	2	59
	0	50	0	20	3	00	0	15	2	45
Av.	0	53.4	0	18.6	3	13.6	0	18	2	57.2

D	585'.
D'	625'.
S—585, 3 min. 13.6 sec.	181' per min.
KS—625, 2 min. 57.2 sec.	212' per min.
K—212/181	1.17.
l	1 min. 30 sec.
D/S+×D'/KS	6 min. 10.8 sec.
w	0.26 cu. yds.
C—7395/10	739.5 cts.
W	600 min.

10 scrapers at $5.50	$55.00
1 snatch	7.50
1 loader	2.25
1 dumpman	1.65
1 foreman	6.55
1 waterboy	1.00
	$73.95

$$R = \frac{73.95}{600} \times \frac{7.68}{0.22} = 43 \text{ cts. cu. yd.} = 84 \text{ cts. static} + 5.9 \text{ cts. per 100-ft. haul.}$$

TABLE VIII.—SCRAPER WORK IN COARSE SAND.

	Waiting and preparing to load.		Loading.		Transporting loaded.		Dumping and turning.		Transporting empty.	
	Min.	Sec.	Min.	Sec.	Min.	Sec.	Min.	Sec.	Min.	Sec.
	1	05	0	37	2	59	1	04	2	48
	3	20	0	17	2	35	1	07	2	06
	4	08	0	14	3	14	1	12	1	56
	1	05	0	12	2	31	0	14	2	26
	1	26	0	12	3	08	0	12	2	00
	1	42	0	18	2	18	0	24	2	03
	1	09	0	15	2	45	1	08	2	43
	0	49	0	11	3	00	1	17	2	00
	1	10	0	18	2	20	1	14	2	26
	0	50	0	13	1	45	1	22	3	00
Av.	1	40.4	0	16.7	2	39.5	0	56.4	2	21

D	585'.
D'	610'.
S—585, 2 min. 39.5 sec.	220' per min.
KS—610, 2 min. 21 sec.	259' per min.
K—259/220	1.18.
l	2 min. 53.5 sec.
D/S + D'/KS	5 min. 00.5 sec.
w	0.26 cu. yds.
C—68.45/9	760.5 cts.
W	600 minutes.

Cost of 9 scraper gang	$68.45

$$R = \frac{760.5}{600} \times \frac{7.9}{0.26} = 38.5 \text{ cts. per cu. yd.} = 14.1 \text{ cts. static} + 1.2 \text{ cts. transportation per 100-ft. haul.}$$

TABLE IX.—WHEEL SCRAPER WORK IN COARSE, DRY SAND, VERY LONG HAUL.

	Waiting and preparing to load.		Loading.		Transporting loaded.		Dumping and turning.		Transporting empty.		Delays.	
	Min.	Sec.	Min.	Sec.	Min.	Sec.	Min.	Sec.	Min.	Sec.	Min.	Sec.
	0	26	0	12	4	07	0	43	3	50	.	..
	0	28	0	15	5	00	0	23	3	46	.	..
	0	46	0	31	4	50	0	15	3	38	0	15
	0	32	0	20	4	49	0	22	3	37	.	..
	0	32	0	30	5	23	0	22	3	43	.	..
	0	22	0	15	5	00	0	21	3	44	.	..
	0	56	0	12	5	02	0	14	3	21	.	..
	0	38	0	15	3	52	0	21	4	04	.	..
	0	24	0	17	4	00	0	41	3	20	.	..
	0	43	0	18	4	17	0	18	3	42	.	..
Av.	0	34.7	0	18.5	4	38	0	24	3	46.5	0	1.5

D = D' —	1040'.
S—1040/4 min. 38 sec.	224' min.
KS—1040/3 min. 46.5 sec.	276' min.
K—276/224	1.23.
l	1 min. 18.7 sec.
D/S (1+1/K)	8 min. 24.5 sec.
w	0.296 cu. yds.
C—7020/9	780 cts.
W	600 min.

9 scrapers at $5.50	$49.50
1 snatch and driver	7.50
1 loader	2.25
1 dumpman	1.75
1 shovelman	1.65
1 foreman	6.55
1 waterboy	1.00
	$70.20

$$R = \frac{780}{600} \times \frac{9.72}{0.296} = 12.6 \text{ cts. cu. yd.} = 5.7 \text{ cts. static} + 3.5 \text{ cts. per 100-ft. haul.}$$

umpman, regularly required for wheel scraper vork, may be dispensed with. The salient features of the Fresno are the speed and ease vith which it can be loaded and dumped nd also its much lower first cost. The capacity f the Fresno ordinarily hauled by three horses s from 7 to 8 cu. ft. as against a capacity of to 10 cu. ft. for the No. 2½ wheelers. As nentioned at the beginning of this paper, in comparing two methods to determine the more economical for the handling of material it boils lown to a question of amount carried and speed of handling. Now, it is evident that of wo methods of practically equal cost of operaion if one is sufficiently more speedy than the ther it will do work more economically even hough handling less material per unit. On the other hand there may come a point where the naul gets so long that the advantage gained in speed of loading and dumping (the time consumed in transit of course assumed as being the same in both instances) will be overcome by the fact that the other method handles more material.

For example, assume, Table XII, two gangs, one composed of five Fresno scrapers and one of five wheel scrapers. For this gang the unit cost per scraper is practically the same, say 900 cts. For each round trip the wheeler loads in 1½ minutes and dumps and turns in ½ minute. The Fresno loads in ½ minute and dumps and turns in ¼ minute. Then in the formula, $y = \frac{C\,D/S\,(l + l/K) + l}{W\,w}$.

TABLE XII.

	Wheelers.	Fresnos.
D	220′.	220′.
S	220′ per min.	220′ per min.
KS	220′ per min.	220′ per min.
K	1	1
l/K	1	1
l (1½ + ½).....	2 min.	(½+¼) 0.75 min.
D/S (1 + 1/K)..	2 min.	2.00 min.
W	600 min.	600 min.
w	⅓ cu. yd.	¼ cu. yd.
C	900 cts.	900 cts.

$$R = 900.\frac{2+2}{600\times\frac{1}{2}} = 18 \text{ cts. per cu. yd.}$$

for wheelers. $= 900.\frac{2+0.75}{600\times\frac{1}{4}} = 16\frac{1}{2}$ cts. per cu. yd. for Fresnos, showing the latter to be more economical.

Or suppose the assumptions are as in Table XIII.

TABLE XIII.

	Wheelers.	Fresnos.
D	440′	440′
S	220′ per min.	220′ per min.
KS	220′ per min.	220′ per min.
K	1	1
l/K	1	1
l (1½ + ½).....	2 min.	(½ + ¼) 0.75 min.
D/S (1 + 1/K)..	4 min.	4 min.
W	600 min.	600 min.
w	⅓ cu. yd.	¼ cu. yd.
C	900 cts.	900 cts.

$R = 900.\frac{4+2}{600\times\frac{1}{2}} = 27$ cts. per cu. yd. for wheelers $= 900.\frac{4+0.75}{600\times\frac{1}{4}} = 28\frac{1}{2}$ cts. per cu. yd. for Fresnos.

Showing that now the wheelers are working the more economically.

The Fresno is not suitable for work including many roots or boulders, and is not generally used with snatch teams for loading. When the ground is too hard for Fresno loading it may not be too hard to be loaded by wheelers and snatch teams, but where the haul is short it is generally much more economical to loosen it with a plow and haul with Fresnos than to use the wheelers. The purpose of this paper is mainly to show comparisons between methods where both or all are possible, but when one and one only can be economic. For hauling the material up heavy grades, such as a railway embankment, the Fresno is much more satisfactory than the wheeler and second only to the small scoop. A peculiar advantage of the Fresno for certain kinds of work is that it can be made to take a heavy or light cut at the option of the driver. In grading lawns, finishing cuts, etc., the Fresno can take a thin slice of an inch or a deep scoop of nearly a foot with equal facility, and is controllable in loading by one hand on the dumping lever.

TABLE XI.

Waiting and preparing to load.		Loading.		Transporting loaded.		Dumping and turning.		Transporting empty.	
Min.	Sec.	Min.	Sec.	Min.	Sec.	Min.	Sec.	Min.	Sec.
1	40	0	15	2	53	0	19	2	23
1	55	0	15	2	33	0	25	2	32
0	42	0	13	2	44	0	25	2	15
2	20	0	30	3	00	0	27	2	43
1	49	0	..	2	30	0	27	1	45
0	53	0	19	1	55	0	18	2	00
1	13	0	14	2	31	0	19	2	41
0	31	0	12	2	39	0	18	2	30
0	39	0	10	2	54	0	22	3	15
0	55	0	16	2	47	0	25	2	40
Av. 1	15.7	0	16.5	2	38.6	0	22.5	2	28.4

D........	600′.
D′........	650′.
S...............	226′ per min.
KS........	262′ per min.
K..............	1.16.
l......	1 min. 54.7 sec.
D/S + D′/KS............	5 min. 7 sec.
C......	787 cts.
w..........	0.30 cu. yds.
W......	600 min.

Cost of 8 scraper gang.........$62.95

$$R = \frac{787}{600}\times\frac{7.03}{0.30} = 30.7 \text{ cts. per cu. yd.} = 8.3 \text{ static} + 3.7 \text{ cts. per 100-ft. haul.}$$

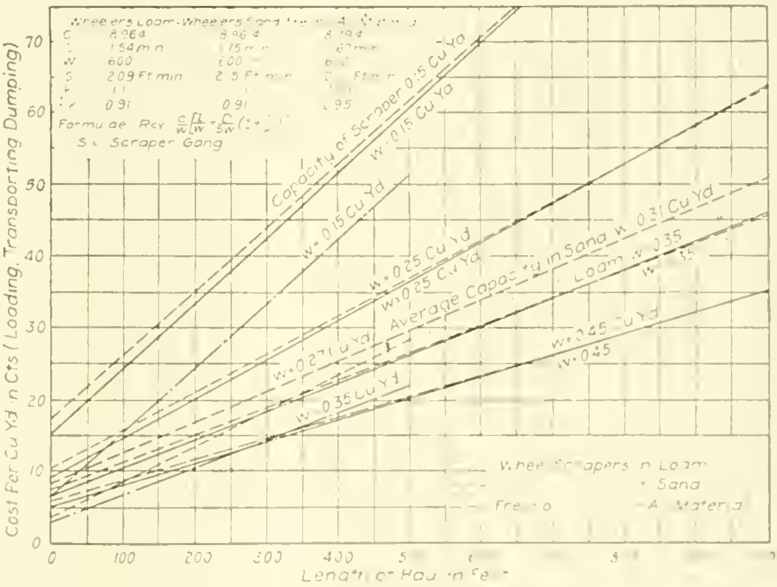


Fig. 12. **Diagram Comparing Economy Up to 275 Ft. Haul of Fresno and Wheel Scrapers.**

TABLE X.

Waiting and preparing to load.		Loading.		Transporting loaded.		Dumping and turning.		Transporting empty.	
Min.	Sec.	Min.	Sec.	Min.	Sec.	Min.	Sec.	Min.	Sec.
0	35	0	15	2	38	0	12	2	30
1	05	0	15	2	33	0	27	1	45
0	34	0	16	2	09	0	11	2	34
0	45	0	13	1	53	0	18	2	49
0	28	0	12	2	35	0	17	2	05
1	00	0	10	2	38	0	25	2	39
0	31	0	10	2	49	0	25	2	15
0	30	0	10	2	20	0	18	2	10
0	35	0	8	2	20	0	22	2	05
0	48	0	12	2	15	0	27	3	03
Av. 0	41.1	0	12.1	2	25	0	20.2	2	23.5

D	600 ft.
D′	650 ft.
S	248 ft. min.
KS	272 ft. min.
K	1.10.
l	1 min. 13.4 sec.
D/S + D′/KS............	4 min. 48.5 sec.
w	0.30 cu. yds.
C	820.7 cts.
W	600 min.

Cost of 7 scraper gang $57.45

$$R = 820.7 \times 6.03 = 27.4 \text{ cts. per cu. yd.} = 5.5 \text{ cts. per cu. yd. static} + 3.65 \text{ cts per 100-ft haul}$$

Table XIV gives results of a large number of cost data on Fresno work in various materials, arranged in the same manner as those on wheelers in the early part of this paper.

Below are shown in detail time studies of various observations, with remarks upon the conditions affecting the results.

The observations in Table XV show the Fresno to have been working very well. The material was a loam and clay, enough of the latter being present to form an ideal track for the scrapers, hard and smooth. The loads were of good size

The static cost here is considerably below the average, which shows at once that there was very little wasted time, and that the work moved smoothly. The transportation cost is somewhat above the average and a glance at *S* and *KS*, the rates of moving, which are low, makes the reason apparent, showing that as a whole the work went smoothly, but slowly. No doubt the general speed of this work could have been considerably increased and still have things move nicely, but it is preferable to have the work run uniformly even at the expense of speed than to rush in transit and wait idle about the loading point.

The observations in Table XVI were made where the material handled was top-soil, rather sticky from recent rains.

In this instance the static cost is far above the average, due to excessive idle time. In the first place the time of "Waiting at Loading

Point" is far too high and arose from a condition very often observed upon scraper work for the existence of which there is no excuse or reason except negligence upon the part of the man in charge. This is allowing the scrapers to get bunched together so that the work becomes spasmodic. Teams are idle at the loading point waiting for the ones ahead to be loaded and when all are en route to the dump the loader is idle until the first arrives again. In consequence the whole speed of action is reduced. It is just as easy to keep teams equally spaced, especially when there are but three or four upon a short haul, as in this case. Then the loader works with regularity, the teams move with uniform speed, do not get bunched, and there is no necessity of wasting time in idle waiting. In the second place the loading time is too great. This was due to the scrapers frequently hitting some hard spot not sufficiently plowed, overturning and having to turn about for reloading. Having the ground well plowed is of prime importance in Fresno work, as it cuts down the actual loading time, and keeps the whole work in motion better. In the third place the time "Turning at Dump" should be reduced one-half There is no necessity of stopping a second in dumping and once the scraper is empt the driver should turn his team right abou without moving from his tracks. Too ofte a long sweep is made, wasting much valuabl time. The transportation charge is above th average due to the slow return from the dum (*K* and *S*), arising also from crowding an the material reduction of speed occurring whe drivers see that teams ahead of them are idl at the loading point.

TABLE XIV.—FRESNO SCRAPER COSTS.

Material.	No. of scrapers.	D.	D'.	S.	KS.	K.	I.	Load, cu. yds.	Cost, cu. yd.	Static cost per cu. yd. in cts.	Transportation. Total.	Transportation. Cost per cu. yd. in cts. per 100-ft. haul.
C. and L.	4	125	165	177	210	1.09	0.39	.240	11.7	2.4	9.3	7.4
S. and C.	3	125	190	188	205	1.09	0.48	.286	11.5	2.6	8.9	7.1
L.	3	100	200	226	236	1.04	0.43	.246	11.3	2.8	8.5	8.5
L.	3	155	155	265	226	0.85	0.44	.250	11.3	2.9	8.4	5.4
L.	3	200	300	226	218	0.97	0.53	.296	14.0	2.9	11.1	5.5
L.	3	130	120	226	234	1.03	0.51	.280	9.4	3.0	6.4	4.9
L.	3	85	105	193	216	1.12	0.51	.280	8.5	3.0	5.5	6.5
L.	2	120	145	264	199	0.75	0.42	.250	11.9	3.1	8.8	7.3
S. and C.	3	155	195	187	189	1.01	0.57	.286	13.5	3.2	10.3	6.6
L.	2	85	105	260	254	0.98	0.49	.280	8.2	3.3	4.9	5.8
C. and S.	3	135	189	198	222	1.12	0.61	.286	11.8	3.4	8.4	6.2
L.	3	112	135	187	238	1.27	0.47	.230	11.8	3.4	8.4	7.5
L.	2	120	140	200	208	1.04	0.54	.250	11.8	3.5	8.3	6.9
S. and C.	3	200	235	212	248	1.17	0.66	.286	13.8	3.6	10.2	5.1
L.	3	112	125	208	283	1.36	0.58	.250	10.3	3.8	6.5	5.8
S. and C.	3	260	310	196	228	1.16	0.64	.250	21.0	4.0	17.0	6.5
C. and S.	3	170	195	226	233	1.03	0.73	.286	13.0	4.1	8.9	5.2
L.	3	140	150	206	235	1.14	0.63	.230	12.7	4.1	8.6	6.1
S. and C.	3	175	210	211	256	1.21	0.78	.286	13.5	4.3	9.2	5.3
S. and C.	3	155	185	218	214	0.98	0.80	.286	13.2	4.4	8.8	5.7
C. and S.	3	130	155	185	208	1.12	0.80	.286	12.5	4.5	8.0	6.1
S. and C.	3	155	180	201	163	0.81	0.82	.286	14.9	4.5	10.4	6.7
L.	3	200	300	204	211	1.03	0.87	.278	18.8	5.0	13.8	6.9
S. and C.	3	245	270	220	212	0.96	1.06	.286	19.1	5.9	13.2	5.4
S. and C.	3	195	235	196	168	0.86	1.15	.286	19.7	6.4	13.3	6.8
S.	3	300	315	214	221	1.05	1.07	.250	24.6	6.8	17.8	5.9
L.	3	120	170	217	195	0.90	1.14	.250	16.7	7.4	9.3	7.7
Min.	2	85	105	177	163	0.75	0.39	0.230	8.2	2.4	4.9	4.9
Av.	3.0	156	192	212	220	1.05	0.67	0.269	13.7	4.0	9.7	6.3
Max.	4	300	315	265	283	1.36	1.15	0.296	24.6	7.4	17.8	8.5

Under "Material" C. and L., clayey loam; S. and C., clayey sand; L., loam; S., sand

D—Loaded haul in feet.
D'—Empty haul in feet.
S—Loaded speed in ft. per min.
KS—Empty speed in ft. per min.
K—Ratio of empty and loaded speed.
I—Time other than that of transportation, in min

TABLE XV.—FRESNO SCRAPER WORK IN CLAYEY LOAM.

Waiting and preparing to load. Min.	Sec.	Loading. Min.	Sec.	Transporting loaded. Min.	Sec.	Transporting empty. Min.	Sec.	Other delays. Min.	Sec.
0	03	0	7	0	53	0	50	0	3
0	12	0	7	0	38	0	35	0	0
0	25	0	8	0	32	0	40	0	0
0	11	0	6	0	41	0	51	0	0
0	9	0	8	0	29	0	41	0	0
0	12	0	12	0	50	0	58	0	15
0	10	0	8	0	55	0	47	0	0
0	7	0	5	0	39	0	48	0	0
0	13	0	15	0	15	0	46	0	7
0	6	0	8	0	43	0	53	0	16
Av. 0	11	0	8½	0	42½	0	47	0	4

D .... 125'
D' .... 165'
S—125'/42½ s. .... 177' per min.
KS—165'/47 s. .... 210' per min.
K—210/177 .... 1.19
I .... 0—23½
D/S + D'/KS .... 1—29½
w .... 0.24 cu. yds.
C—35.75/4 .... 894 cts.
W .... 600 min.

4 scrapers and drivers at $7.20	$28.80
1 loader	1.95
1 foreman	4.00
1 waterboy	1.00
	$35.75

$$R = \frac{894}{600} \times \frac{1.88}{0.24} = 27.1 \text{ cts. per cu. yd.} = 2.4 \text{ cts. static} + 7.4 \text{ cts. per 100-ft. haul.}$$

TABLE XVI.—FRESNO SCRAPER WORK IN HEAVY TOPSOIL.

Waiting at loading point. Min.	Sec.	Loading. Min.	Sec.	Transporting loaded. Min.	Sec.	Turning at dump. Min.	Sec.	Transporting empty. Min.	Sec.
0	40	0	40	0	23	0	14	0	46
0	37	0	14	0	32	0	14	0	50
0	15	0	8	0	37	0	20	0	50
0	50	0	9	0	36	0	25	0	50
0	20	0	15	0	38	0	22	0	65
Av. 0	32.4	0	17.2	0	33.2	0	19	0	52.2

D .... 120'
D' .... 170'
S—120'/33.2 sec. .... 217' per min
KS—170'/52.2 .... 195' per min.
K—195'/217 .... 0.9
I .... 1 min. 08.6 sec.
D/S + D'/KS .... 1 min. 25.4 sec.
w .... 0.25 cu. yds.
C—29.25/3 .... 975 cts.
W .... 600 min.

Cost of 3 scraper gang. $29.25

$$R = \frac{975}{600} \times \frac{2.57}{0.25} = 16.7 \text{ cts. per cu. yd} = 7.4 \text{ cts. per cu. yd static} + 7.7 \text{ cts. per 100-ft. haul.}$$

In the work for which the time study, Tabl XVII, was made the material was well plowe top soil in almost ideal condition for Fresn work.

Here both static and transportation charge are below the average. A more unifor spacing of scrapers was observed with the result that the idle and loading times were materially reduced; and the rate of transportatio was very satisfactory accounting for the lo transportation charge.

To secure cost diagrams, Fig. 8, for Fresn scraper work, average values may be substituted in the general transportation formul given above, where: C = 879 cts.; I = 0.6 mins.; W = 600 mins.; S = 212 ft.; K = 1.05 1/K = 0.95, load and haul being variable.

The costs above given do not include anything for overhead charges, superintenden (above foreman), preparatory charges, offic expenses, contractor's profit, etc. This refer to both classes of scrapers. These values ar plotted on the accompanying chart, from whic with conditions approximately as assumed th cost of such work for various lengths of hau may be read directly.

When Fresno scrapers are loaded fro plowed ground it is infinitely easier to loa when dragging across than lengthwise of th furrow. Double plowing is often economical The dumping operation should be accomplishe by a quick, sharp lift on the handle, and preferably on a down grade. When the ground i very well loosened the driver can do his ow loading as well as dumping. The path to th dump must be reasonably free from obstructions, else the scrapers may dump themselve without intention on the driver's part.

GENERAL HINTS ON ALL SCRAPER WORK.

(1) Be sure to use the right kind o scraper. A Fresno with three mules is economical up to about 275 ft. of haul as agains wheel scrapers with 2 mules, when it can loa readily. Where the ground is full of roots us wheelers.

To drivers:

(2) Report any case of bad fitting harnes to the foreman immediately. Don't let th team drag you by the reins. You are supposed to be able to walk as far as a loade team.

To foreman:

Make a personal detailed inspection of eac mule's harness the first thing in the mornin and at noon, and report any case of ill fittin harness to the timekeeper on his next round Foremen will be held responsible for allowin any mule to work with badly fitting harness.

(3) See that each scraper is fully loaded The cost of plowing is less than 1 ct. per cubi yard, which is less than the cost of lettin scrapers work when only partly loaded.

(4) In loading the scraper when it is onc full of earth do not let the mules try to pul it any farther and overload it. The last tw seconds of drag against the dead weight o earth are mule killers.

(5) On all scraper work drivers are required to walk at all times when the scraper i loaded and they are to walk at all times wit the Fresno scraper, whether loaded or empt With wheeler scraper work drivers should rid on the scraper when it is empty. In steppin on or off of the scraper be sure not to dela the team in any way.

(6) In dumping wheel scrapers try not t dump when the mules are on ground that i lower than the scraper, as by doing this i brings a tremendous load on the mules' necks.

(7) So direct the work that the loaded teams will have the shortest haul and th empty teams if necessary may have a muc longer haul, but in no case should the empty haul be unnecessarily long. It is better to let the mule team stand still to rest than to let

cover unnecessary ground. This seems like simple rule, but its violation has often been bserved on several different jobs.

(8) See that the scrapers are spaced as even a distance apart as possible. This will make the work lighter on the mules, easier on the drivers and will tend to avoid confusion.

(9) The loaded scraper should always have the right of way as against the unloaded scraper.

(10) Whenever a scraper gets stuck or is in any trouble don't lose any time before notifying the foreman and sending for help. The snatch team is employed for the purpose of helping the scrapers at all times and in all possible ways.

(11) Be sure not to have too few scrapers on a long haul and too many scrapers on a short haul; see that every scraper is busy all the time; see that the loader and snatch teams are busy all the time; in short, that each unit of the work is contributing its maximum effort to the accomplishment of the whole.

Figures 9 to 11 show Fresno scrapers in operation and Fig 12 is a diagram showing that Fresno scrapers are more economic than wheel scrapers up to 275 ft. haul.

The observations and time studies, with many of the conclusions and all of the tables contained in this paper were made by Mr. Allan C. Haskell, of Construction Service Co. in the course of several extended investigations in the cost and methods of this class of work for various clients.

TABLE XVII.—FRESNO SCRAPER WORK IN WELL LOOSENED TOP SOIL.

	Waiting at loading point.		Loading.		Transporting loaded		Dumping and turning.		Transporting empty.	
	Min.	Sec.	Min.	Sec.	Min.	Sec.	Min.	Sec.	Min.	Sec.
	0	10	0	6	0	32	0	11	0	31
	0	15	0	6	0	34	0	8	0	32
	0	16	0	6	0	40	0	10	0	31
	0	15	0	7	0	33	0	9	0	33
	0	12	0	5	0	34	0	12	0	32
	0	15	0	5	0	27	0	13	0	33
	0	9	0	4	0	36	0	12	0	34
	0	13	0	5	0	30	0	15	0	25
	0	25	0	8	0	44	0	10	0	27
	0	8	0	7	0	36	0	10	0	30
Av.	0	13.8	0	5.9	0	34.6	0	11	0	30.8

'.........................130′
'.........................120′
..........................226′ per min.
S.........................234′ per min.
—231/226 ....1.035
..........................9—30.7 sec.
/S + D′/KS................1—05.4 sec.
..........................0.28 cu. yds.
—29.75/3 ....92 cts.
V.........................600 min.

Cost of 3 scraper gang........$29.75

$t = \frac{992}{600} \times \frac{1.6}{0.28}$ 9.4 cts. per cu. yd. = 3.0 cts. static × 4.9 cts. per 100-ft. haul.

Water Works

The Individuality of Water Works Systems.

The fact that here has been a popular tendency in the past to measure the reasonableness of rates of one public utility plant, or even to fix rates for a new plant, by a comparison of those in force in other cities seems to warrant a little consideration of the difference between various plants of like kind and between the conditions under which they operate. These differences seem generally far from being fully appreciated except by the comparatively few who have had something to do with a considerable number of such properties and had occasion to make comparisons between them. In his paper before the recent annual convention of the American Water Works Association, Mr. W. E. Miller, of the Joint Engineering Department of the Wisconsin Tax and Railroad Commissions, discussed this matter and presented numerous data indicating the individuality of public utilities. The portions of Mr. Miller's paper which relate to water works are here reprinted.

The popular comparison of rates of different water or other utility plants appears to be merely a superficial one, due to the lack of knowledge and appreciation of the great differences that may exist even in cities of similar size. Certain water works franchises have specified that the rates to be charged by the grantees under those franchises should not exceed the average of those in certain other cities named therein.

In the field of water works construction there are many different factors and combinations of factors which enter into the costs of plants. The mere costs of prospecting for and developing water supplies for relatively small communities have often been relatively, if not actually greater than in the cases of much larger cities, particularly so when the losses involved in necessary changes from one source to another, if not to a third and even a fourth source, are considered.

Topography and geology are influential factors in the costs of water plants. Even climate is an important consideration since it affects the depths at which mains and services are to be laid and the character of other features. One does not have to travel very far in certain sections to find standard trench depths differing quite remarkably. For example, at Chippewa Falls, in the west central part of Wisconsin, the franchise granted to the water company required a depth of 9 ft. of cover, which involved cutting into a considerable amount of solid rock. At Hurley, on the north line of Wisconsin, and Ironwood, in upper Michigan, the water mains are laid in a trench only 5 ft. deep and it is understood that very little, if any, trouble through freezing of mains has occurred. Heavy snow covering throughout the winter is given as the reason for the comparatively shallow penetration of frost in those places. Milwaukee's specifications for pipe laying call for a depth of 6½ ft. to the center of pipe. Within the state of Wisconsin alone the standard depths of cover on water mains in different cities have been found to range from 4½ ft. to 9 ft.

It seems needless to say that the character of material to be excavated and the cost of excavating it have also varied between wide limits, covering everything from a merely damp loamy sand, requiring no use of picks, sheet piling or pumps, to solid rock, involving drilling and blasting, or to wet sand and gravel requiring tight sheathing and much pumping.

Conditions very favorable to low cost of pipe laying predominate in some cities while in others the work is very difficult almost throughout. One small city in Wisconsin within the writer's knowledge has found it impracticable to finance the construction of a water works solely because of the fact that the pipe laying would involve the expense of almost continuous excavation of solid rock.

In discussing the distribution system, which forms so large a part of the cost of every water works plant, it is also of interest to note how various plants differ as to average size of pipe in mains. This, as well as the cost of laying, materially affects the total investment. Some water pipe systems contain a considerable proportion of small wrought iron and steel pipe, ranging from ¾ in. to 3 in., running longitudinally with the streets and classed as mains, although serving only private consumers, while other cities have nothing smaller than 4-in. or even 6-in. pipe.

Data for a number of Wisconsin water plants, showing for each case the population by 1910 census, the total mileage of mains and the average pipe diameter are given in Table I. The average size was determined by multiplying each diameter by the length of that size and dividing the sum of those products by the total length of all sizes.

In addition to the data given in Table I the following similar statistics for certain water plants outside of Wisconsin are available:

Place.	Population	Miles of pipe	Av'ge Diam., Ins.
Chicago, Ill.	2,185,283*	2,362†	8.12
Indianapolis, Ind.	233,650*	264.15§	8.28

*1910 †1911 §1911.

The foregoing figures were, in most cases, compiled from statistics furnished by the plants for the year ending June 30, 1913. In a few instances statistics on pipe mileage for that year were not available and those of some-

TABLE I.—AVERAGE SIZES OF WATER MAINS IN VARIOUS WISCONSIN CITIES.

Class A Plants.

Place.	Population	Miles of mains.	Av'ge Diam., ins.
Private ownership—			
Ashland	11,594	30.70	6.87
Beloit	15,125	27.56	6.38
Chippewa Falls	8,893	.....	...
Green Bay	25,236	96.74	1.13
Janesville	13,894	32.55	5.60
Marinette	14,610	33.66	5.22
Merrill	8,689	19.92	5.49
Oshkosh	33,062	60.31	6.23
Racine	38,002	76.43	7.06
Superior	40,384	61.82	7.95
Municipal—			
Appleton	16,773		
All sizes		29.72	5.57
Cast iron only		22.62	6.79
Eau Claire	Pipe data not available.		
Fond du Lac	18,797	33.82	5.81
Kenosha	21,371	41.93	6.12
La Crosse	30,417	63.97	8.22
Madison	25,531	51.13	5.18
Manitowoc	13,027	24.63	6.94
Milwaukee	373,857	[illegible]	8.61
Sheboygan	26,398	68.50	5.69
Watertown	8,829	20.59	6.46
Waukesha	8,740	25.86	6.70
Wausau	16,560	31.92	5.33

Class B Plants

Place.	Population	Miles of mains.	Av'ge Diam., ins.
Private—			
Antigo	7,196	13.97	5.46
Beaver Dam	6,758	16.67	6.17
Mellen	1,833	3.68	6.30
Menomonie	5,036	10.91	5.65
Oconto	5,629	11.48	5.28
Ripon	3,739	11.63	5.92
Stevens Point	8,692	16.01	6.09
Washburn	[illegible]		
All sizes		8.43	5.50
4-in. and larger		6.67	[illegible]
Whitewater	3,224	8.76	6.83
Municipal—			
Baraboo	6,324	11.36	6.20
Berlin	4,636	9.58	6.57
Columbus	2,523	6.86	[illegible]
Edgerton	[illegible]	[illegible]	[illegible]
Fort Atkinson	[illegible]		
4-in. and larger		8.78	[illegible]
All sizes		10.28	[illegible]
Grand Rapids	6,521	[illegible]	[illegible]
Hudson	[illegible]	8.78	4.69
Kaukauna	4,717	10.46	6.88
Lake Geneva	[illegible]	7.72	[illegible]
Lancaster	[illegible]	[illegible]	4.44
Marshfield	5,783	8.99	7.72
Menasha	6,081	13.29	7.87
Monroe	4,410	8.98	5.39
Neenah	5,734	13.75	6.53
New Richmond	1,988	3.22	5.63
New London	3,383	7.87	5.70
Platteville	[illegible]	11.65	4.75
Portage	5,440	18.37	4.21
Port Washington	3,792	17.17	6.10
Rhinelander	5,637	15.28	5.92
Rice Lake	3,968	9.76	4.73
Richland Center	[illegible]	7.68	5.05
River Falls	[illegible]	5.16	5.65
Shawano	[illegible]	5.94	5.56
South Milwaukee	6,092	7.72	6.17
Sparta	[illegible]	12.95	4.93
Stoughton	4,761	13.37	4.43
Sturgeon Bay	4,262	4.84	9.43
Tomah	3,419	7.63	4.74
Tomahawk	2,907	5.31	5.49
Two Rivers	4,850	10.05	4.61
Waupaca	2,789	9.71	5.27
West Allis	6,645	27.9	6.56
Oconomowoc	3,054	8.74	6.88
Plymouth	3,094	9.83	4.89

what earlier dates were used, being taken from inventories and valuations.

In those cases where much pipe smaller than 4 ins. is used it is evident that the fire service is either not coextensive with the pipe system or it is of a lower order of excellence than in those cases where no such small pipe is found.

The question sometimes arises as to how marked is the tendency for the average diameter of distribution mains of a water plant to increase with the total mileage or with the population of the cities supplied. In order to most clearly show that tendency so far as the foregoing figures indicate its existence those values have been put in graphical form as shown by the diagrams Fig. 1 and Fig. 2. The former shows the various cities arranged in the order of magnitude of their population, the second in the order of total pipe mileage.

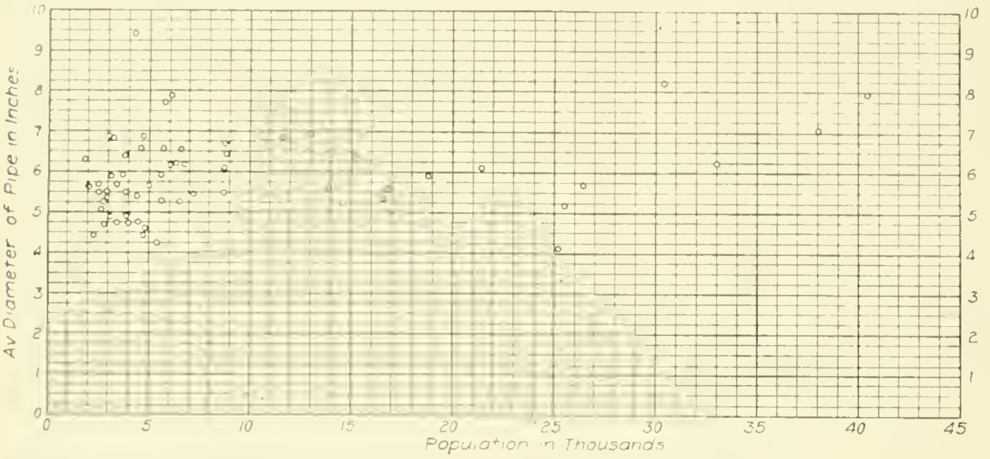


Fig. 1. Diagram Showing the Variation in Average Diameters of Mains in 64 Wisconsin Water Systems—Cities Arranged In Order of Population.

The great diversity of average pipe diameters can hardly be fully accounted for by differences in local conditions alone but are doubtless due in large measure to differences in personal ideas of those who have had to do with the development of these systems. It is also to be remembered that these several plants will certainly not show the same degree of adequacy under their respective sets of operating conditions and requirements. They are all, doubtless, fully adequate for the ordinary daily commercial service, but the capacity for good direct pressure fire service in addition is low in those cities where average pipe diameters are small. In two cities of similar size and having very similar pipe systems the fire service will be poorer in that one using the greater amount of water in the general commercial service.

It can scarcely be contended that the diagrams Figs. 1 and 2, establish the existence of a tendency for the average diameter of mains in a system to increase as the mileage increases or as the city grows in population. The developments of single or individual systems would be much more reliable indications of the natural changes in average pipe sizes, as the great diversity of ideas of those in charge of extensions of various systems and the great diversity of other local conditions would largely be eliminated.

Figure 3 shows the change in average pipe diameter with increase in total mileage in two water pipe systems in Wisconsin, those of Madison and Milwaukee.

In order to indicate the approximate effect of difference in average pipe diameter upon the average cost per foot of all mains in a system, the diagram Fig. 4 has been prepared from unit prices used in 1911 valuation of the Milwaukee water works That valuation was made by the engineering staff of the railroad commission of Wisconsin for the commission's use in determining equitable rates. The diagram shows values per foot of mains including material and labor costs. These unit values are, of course, not applicable in valuation of other pipe systems where conditions are different. The diagram merely indicates approximate relative unit values on different sizes, or average sizes, under certain specific conditions.

Were all conditions except average pipe diameter similar in Madison and Milwaukee, Figs. 3 and 4 would indicate that the cost of the Milwaukee system per mile or per foot would exceed that of Madison by about 60 per cent.

Among the numerous factors entering into the problem of the pipe system and into the difference existing between the several systems named above, both as to average diameter and cost per foot or other unit of length, are: (1) Depth of cover required to prevent freezing; (2) character of soil as regards difficulty of excavating it; (3) rates of cost for materials and labor; (4) average and maximum present rates of demand of private service; (5) provision or lack of provision of a temporary surplus of capacity to take care of future extensions into new territory; (6) provision for increased demand in existing territory; (7) provision for reduced carrying capacity by incrustation or tuberculation; (8) character of fire service to be provided; (9) location of pumping plant with respect to center of distribution; (10) permissible total loss of head by friction (affected by topography of territory), etc.

A little reflection on the number of factors that affect the design, development and cost of a water pipe system, and on the range of values any one factor may have in different cases shows that it is but natural for such systems to possess decided individuality.

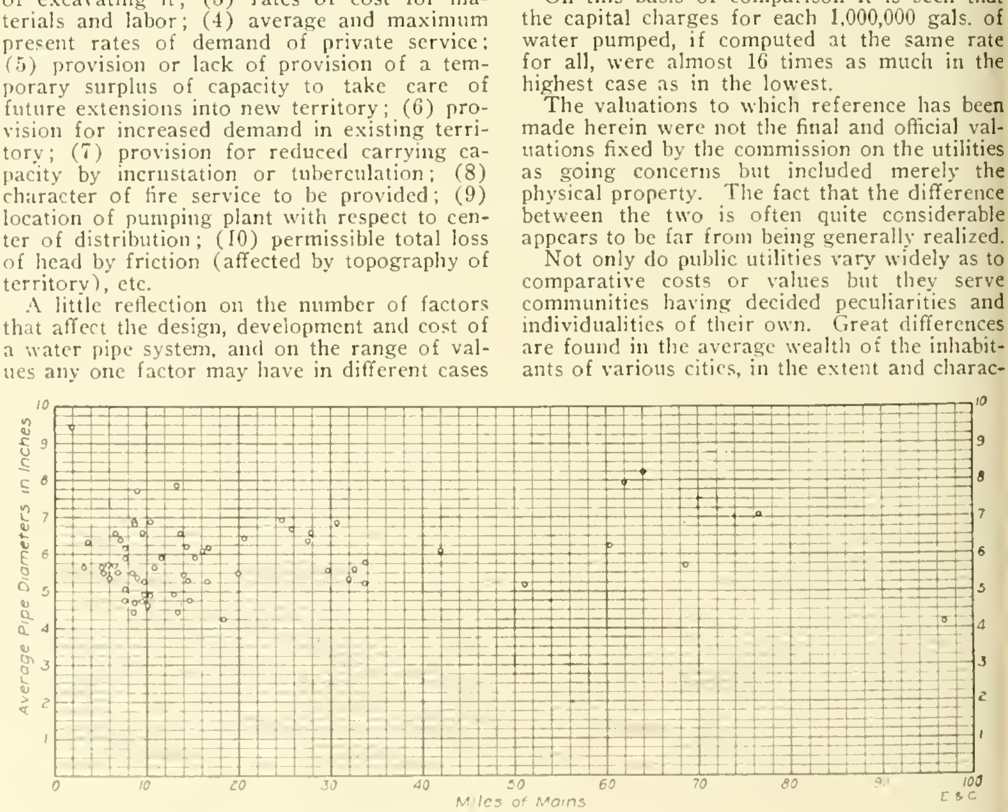


Fig. 2. Diagram Showing the Variation In Average Diameters of Water Mains in 64 Wisconsin Cities—Cities Arranged in Order of Pipe Mileage.

The entire distribution system of a public water works property often represents two-thirds or even a greater proportion of the total plant value, yet in a number of cases it falls to materially less than one-half the total. Such differences are, in part, due to lack of uniformity in the matter of ownership of service pipes and meters, these being generally counted as parts of the distribution system in cases where they are installed by and at the expense of the water works, instead of by property owners or tenants, as is the custom in some cities.

While the foregoing and other facts show that very little is to be gained by merely comparing total water plant investments or valuations in different cases on any basis, it is considered of some interest to note how such total values do vary.

Thirty-three of the Wisconsin water works plans of which physical valuations have been made by the engineering staff of the railroad commission of Wisconsin for the commission's use and consideration in rate or purchase cases compare as follows on the "per capita" basis:

2 plants showed values betw.	$14.70	and	$15.00
14 plants showed values betw.	15.00	and	20.00
10 plants showed values betw.	20.00	and	25.00
4 plants showed values betw.	25.00	and	30.00
2 plants showed values betw.	30.00	and	35.00
0 plants showed values betw.	35.00	and	40.00
1 plant showed a maximum of	40.96		
33 plants showed an average of	$21.34		

The same plants compared on the basis of total physical valuation per service connection show the following relative amounts:

2 plants had values between.	$ 75.00	and	$100.00
12 plants had values between.	100.00	and	150.00
10 plants had values between.	150.00	and	200.00
5 plants had values between.	200.00	and	250.00
4 plants had values between.	250.00	and	300.00
33 plants averaged			$171.91
22 plants in the largest groups averaged..			$146.95

When compared on the investment per 1,000,000 gals. pumped during a year these plants appeared as follows:

5 plants furnished no pumping statistics.			
3 plants had values between....	$ 380	and	$ 400
4 plants had values between....	400	and	500
9 plants had values between....	600	and	703
5 plants had values between....	800	and	1,000
6 plants had values between....	1,000	and	1,300
1 plant had a maximum value of	6,028		

On this basis of comparison it is seen that the capital charges for each 1,000,000 gals. of water pumped, if computed at the same rate for all, were almost 16 times as much in the highest case as in the lowest.

The valuations to which reference has been made herein were not the final and official valuations fixed by the commission on the utilities as going concerns but included merely the physical property. The fact that the difference between the two is often quite considerable appears to be far from being generally realized.

Not only do public utilities vary widely as to comparative costs or values but they serve communities having decided peculiarities and individualities of their own. Great differences are found in the average wealth of the inhabitants of various cities, in the extent and character of the industries located therein, in the topography and geology of the land on which they are built and in other aspects. All these decisively affect the demands on the utilities and therefore their operating expenses, and necessary rates.

A comparison of the pumping expenses alone of 17 of the larger Wisconsin water plants, excluding Milwaukee, showed a range of from $10.38 to $32.32 per 1,000,000 gals. during the year ending June 30, 1913. The head pumped against, not including suction lift, varied from

100 ft. to about 230 ft. There was apparently but little relation between the pumping pressure and the cost per 1,000,000 gals. pumped. Some of the plants in Wisconsin pump against a domestic pressure of as low as 70 ft. head, while just across the state line is one (in Duluth) which raises water (in three lifts) from Lake Superior to an elevation of about 900 ft.

The extra expenses of pumping the same water twice, once from deep wells to a low reservoir or from a lake or stream to a purification plant, then to the city mains and the expenses of purification are material elements in the total cost of service of some plants and are not present in that of others.

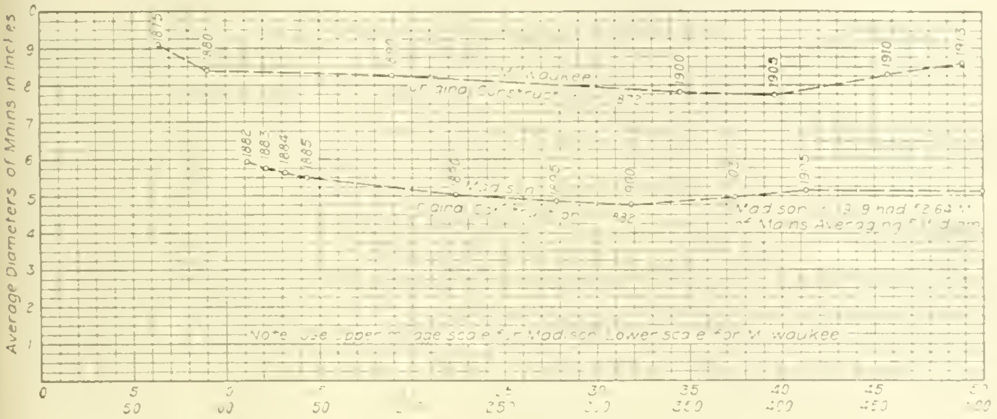


Fig. 3. Diagram Showing Changes in Average Diameters of Water Mains with Increase in Mileage in Milwaukee and Madison, Wis.

Some plants are found to be furnishing service to substantially all, and others to less than half of the inhabitants of their respective communities. Even if all cities were built equally compact, this variation in proportions of population served would materially affect the number of consumers per mile of mains, or, in other words, the number of feet of mains for each consumer in the cases of different utilities.

Most cities show some growth. In certain cases this has been positive, permanent and rapid, in others it has been very slow, even to a negative growth or reduction. Rapidity of community growth affects the depreciation accruing to the utility through obsolescence and inadequacy, causing replacements and enlargements before existing facilities have become worn out.

One of the very important differences between the rates of various water utilities is in the manner in which the total expense has been divided between the public and the private service. By the public service is meant that included in the hydrant rentals. By private service is meant all the remainder, which is sometimes designated as the general service and subdivided into classes termed domestic, commercial and industrial service.

Until a very few years ago the determination of hydrant rentals seems to have rarely if ever been put upon a scientific and equitable basis. Such charges were generally, if not always, fixed by a process of bargaining between municipal authorities and the owners or managers of the utilities. The latter naturally got what they could of their total revenues in the form of hydrant rentals and endeavored to spread the balance of the total expense of operation and fixed charges over the general service.

There is probably no need of reproducing here any of the abundant published data showing how greatly the hydrant rentals have varied in different cities. That such great differences exist seems to be quite widely known. Obviously, if a waterworks is to have a fair and proper gross income, it must make up from the private consumers, or water takers, whatever amount of fair revenue it fails to get in return for its municipal hydrant service. Rates for the former are clearly dependent in part upon whether or not the latter service yields its just proportion of the total expense.

These and numerous other considerations which might be mentioned, all demonstrate quite clearly that every water plant has its own peculiarities and individuality. The best interests of both the utility and the public it serves, demand that it be considered separately and independently of all others, except, perhaps, when all the differences in local conditions are fully understood and due allowances are made therefor. The law of averages has its uses but it certainly is not applicable in determining reasonable rates for one plant by those of others.

In making comparisons between various water plants, for example, it is to be remembered that the statistics of each plant are continually changing and in some cases much more rapidly than others. At any given time the business of each plant in any list that may be taken is in a particular stage of its development and that of each plant is likely to be quite different from all the others. In the writer's judgment these considerations all demonstrate quite clearly that each plant is to be considered upon its own particular conditions and merits as a special case having decided individuality.

Specifications for Driving 12-in. Wells in Water Bearing Sands at San Diego, Cal.

The following matter is abstracted from the specifications recently drawn up by the department of water of San Diego, Cal., Maj. Herbert R. Fay, superintendent, to govern the bidding on and sinking of 13 12-in. wells in the water-bearing sands in Mission Valley. These specifications are out of the ordinary in that they serve largely to minimize the gamble incident to the well drilling business as it is usually conducted. We have commented editorially upon this phase of the subject elsewhere in this issue. In this abstract of the specifications only the purely engineering features are touched upon, general provisions being omitted.

The location of the wells is indicated upon a map prepared for that purpose. The 13 wells will be 12 ins. in diameter and will be drilled to a depth of approximately 80 ft. from the surface.

The contractor will furnish all well casings, shoes and starters; the well casing will be 12-in. No. 12 gage double well pipe with a 12-in. No. 12 gage riveted starter, 21 ft. long, two ply for 15 ft. and three ply for 6 ft. with a 12x6x¾ in. shoe. The well casing will be of hard steel, especially adapted for the purpose for which it is to be used, and of approved design.

The contractor will furnish first class drilling machinery and competent men to operate it and will shove, or drive the well casings through the water-bearing sand into the clay, or other solid material beneath in order that all loose strata will be prevented from entering the well from around the shoe. After stopping the 12-in. casing the contractor will drill sufficiently far in advance to form a sand pocket and proper submergence for an air pump, or other appliance, this depth to be determined by the hydraulic engineer, department of water.

The contractor will drill the well true and straight and it must permit of a 9⅝ in. inside diameter casing being let from the top of the well to the bottom without forcing and withdrawn without the least difficulty.

Upon completion of the well the contractor will perforate the casing with a star, or other approved design of perforator of approved design. All water-bearing strata will be perforated with four rows of perforations, perforations to be in such manner as to permit of a minimum amount of sand with a maximum amount of water.

Log of Well.—The well driller is to furnish small bags with strings on same, size of the bags to be, when flat, 5 ins. long by 5 ins. wide and samples of material from which each well is dug shall be placed in the bags and tagged at what depth the material was excavated from the well. Not less than one sample shall be taken for every 10 ft. of depth of well. In passing from one class of strata to another, a sample shall be taken at the beginning of the strata. A log of the well is to be furnished by the contractor, showing accurately the location of the various strata from the surface of the ground and also the elevation of the top of the pipe above the surface of the ground and the location of each series of perforations, the number of perforations and the depth they are from the top of the pipe.

Local Data.—In submitting bids the contractor will have advantage of any information which may be in the files of the department of water in order to assist him in giving an intelligent bid. Three wells were drilled in Mission Valley in which there was fine sand and small gravel down to a depth of 80 ft., from 80 ft. down to 186 ft. was shale. One well had a bed of gravel from 22 ft. to 46 ft. Another well had 8 ft. of gravel from 72 ft. to 80 ft., overlaying the shale bed. The third well had practically no gravel, just fine sand. The wells under these specifications will be drilled in a little coarser sand than those in the lower valley.

BIDDING REGULATIONS.

The contractor shall bid according to these specifications and these specifications only. No bid will be considered in which the contractor promises to give an alternative for which he will do the work. The following is the schedule upon which contractors shall bid per foot:

(a) From 0 ft. to 100 ft. deep, price per foot for sand, silt, dirt, etc.

(b) From 0 ft. to 100 ft. deep, price per foot for gravel.

(c) From 0 ft. to 100 ft. deep; price per foot for boulders or rock.

(d) From 0 ft. to 100 ft. deep; price per foot for shale.

(e) Price per foot beyond 100 ft. in depth.

(f) Price for cleaning and testing each well.

(a) Includes all material that can be easily worked without undue pounding and shall in-

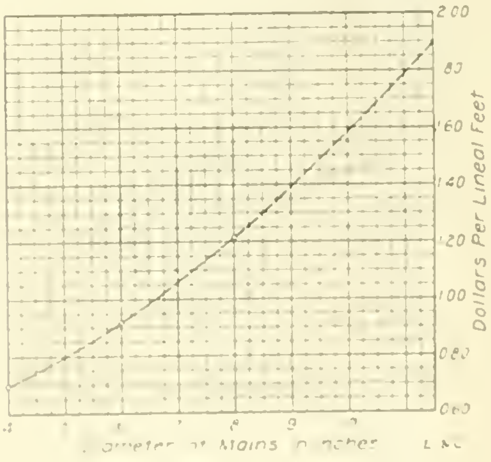


Fig. 4. Diagram Showing Values Per Lineal Foot for Mains of the Milwaukee Water Works.

(As estimated by engineers [illegible] Railroad Commission, 191[illegible]) clude all gravel of small character up to and including 1½ ins. in diameter.

(b) Includes all gravel above 1½ ins. in diameter to what is known as boulders.

(c) Boulders include such material as needs breaking in order to be brought to the surface by ordinary drilling processes and are presumed to be, where in question or doubt, of 6 ins. or larger.

(d) Shale will be considered as a heavy claylike substance which will not readily break under the action of the churn drill and underlies the bed of the valley. All questions in regard to shale will be settled by the hydraulic engineer and in no cases will compacted silt be regarded as shale, it will come under class (a).

Each well will be thoroughly cleaned and pumped out and tested to the satisfaction of the hydraulic engineer.

Mr. H. A. Whitney is hydraulic engineer of the San Diego water department.

Some General Plans and Preliminary Engineering Studies for the Development of the Hetch Hetchy Water Supply for San Francisco.

The construction of the Hetch Hetchy water supply system for the city of San Francisco is an undertaking of such magnitude that it will require the most careful preliminary study to insure the completion of the various units of the project at the proper time. It is such a short time since the Hetch Hetchy grant was secured by the city that at present there can be given only a very general idea of the engineering steps to be undertaken. It is estimated that if the work is carried through on a business basis, with no interruptions from the usual causes which impede municipal work, the Hetch Hetchy water should be delivered at the city inside of seven years. It is our plan to report the features of the design and construction of this important water supply development as fast as authoritative information becomes available for that purpose. What may be termed the engineering preliminaries are here discussed from information taken from a recent address before the Commonwealth Club of San Francisco by Mr. M. M. O'Shaughnessy, city engineer of San Francisco. The accompanying map, obtained from another source, is presented to show the tentative location of the Hetch Hetchy aqueduct, the country traversed by it, the location and extent of watersheds and the sites of projected reservoirs. Some of the watersheds controlled by the Spring Valley Water Co., which now supplies the city with water, are also shown in the map.

Having the right under the Raker bill to impound water in Hetch Hetchy valley, Lake Eleanor, and Cherry Creek watersheds, by constructing dams on these reservoir sites the city can store in the Sierras a total of 689,700 acre-ft., or approximately 225,000,000,000 gals. of water.

In addition to the reservoir sites at Hetch Hetchy, Lake Eleanor, and Cherry Creek, there are many other feasible storage sites on the upper reaches of the San Francisco drainage area which brings the grand total storage on the drainage area controlled by the city to 836,800 acre-ft.

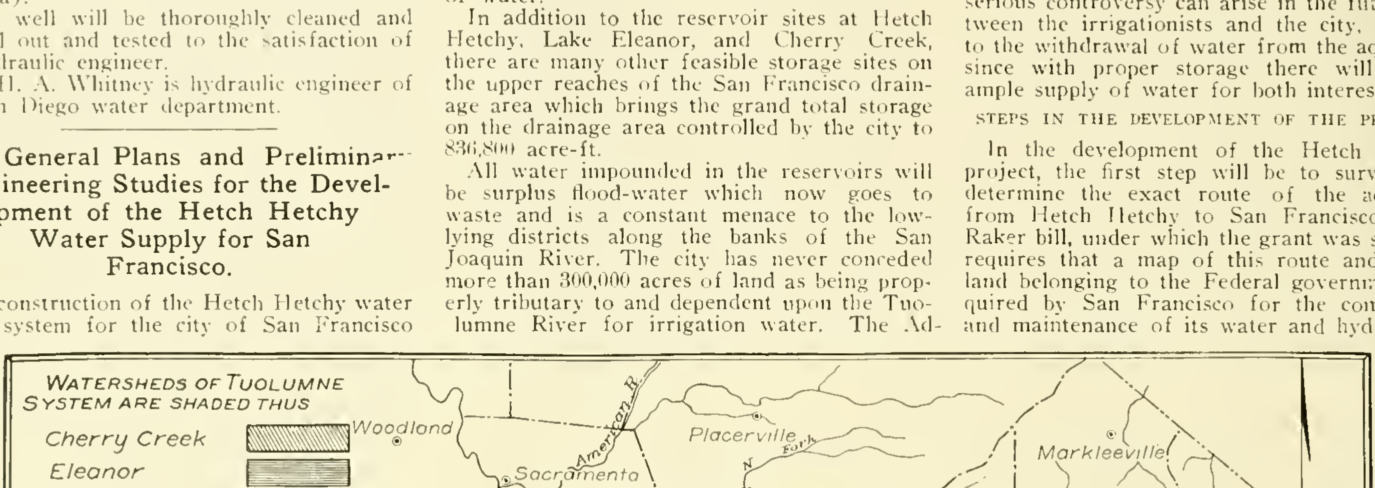
All water impounded in the reservoirs will be surplus flood-water which now goes to waste and is a constant menace to the low-lying districts along the banks of the San Joaquin River. The city has never conceded more than 300,000 acres of land as being properly tributary to and dependent upon the Tuolumne River for irrigation water. The Advisory Board of Army Engineers, however, states that when the dams are completed and the reservoirs filled, there will be a daily supply of over 100,000,000 gals. for San Francisco, and surplus sufficient to irrigate 400,000 acres of land. Considering the availability of the Merced River to the south and the Stanislaus River to the north, and the amount of land that should be properly tributary to these streams when their run-off is stored, there need be no fear that the area to be irrigated from the Tuolumne will ever exceed 300,000 acres. By the time that San Francisco needs 400,000,000 gals. per day from its Tuolumne supply, the irrigation districts can have developed 635,000 acre-ft. of storage from the Tuolumne watershed alone, the waters for which will come almost entirely from drainage areas not controlled by San Francisco.

The foothill storage of 168,000 acre-ft., together with the 167,000 acre-ft. of storage on the upper Tuolumne River watershed (outside of the area controlled by San Francisco and below Jawbone Creek—as provided in the Raker bill) gives a grand total storage of 635,000 acre-ft. available to the irrigation districts. In addition to this there is a very large storage site about five miles southeast of the proposed Davis reservoir of the Turlock irrigation district. This is known as the Dry Creek reservoir site; it has a capacity of 325,000 acre-ft. The city has, then, as the available storage on the San Francisco drainage area 836,800 acre-ft. and outside of the San Francisco area and available for use by the irrigation districts, 960,000 acre-ft.

From the foregoing it is evident that no serious controversy can arise in the future between the irrigationists and the city, relative to the withdrawal of water from the aqueduct, since with proper storage there will be an ample supply of water for both interests.

STEPS IN THE DEVELOPMENT OF THE PROJECT.

In the development of the Hetch Hetchy project, the first step will be to survey and determine the exact route of the aqueduct from Hetch Hetchy to San Francisco. The Raker bill, under which the grant was secured, requires that a map of this route and of all land belonging to the Federal government required by San Francisco for the completion and maintenance of its water and hydro-electric works, be filed with the Secretary of the Interior before any permanent construction work is begun. A 9¼-mile wagon road will be built from Hog Ranch to the site of the dam. This road will be 17 ft. wide, macadamized, and have a maximum grade of 4 per cent. From Smith's station, on the old Oakdale-Yosemite stage route, a good road 22 miles in length has already been constructed to Hog Ranch. With a few alterations and repairs, this road will meet the present needs. Field headquarters for an engineering force will be establishd near the Hetch Hetchy damsite. Trails will be built and telephones installed to connect these headquarters with all operating divisions of the work as it progresses. A construction railroad to the damsite will also be built, but the exact route for this has not yet been determined. When this preliminary work shall have been completed, there will be erected a temporary power-plant



Map Showing Tentative Location of the Hetch-Hetchy Aqueduct, of the Proposed Additional Water Supply for San Francisco, and the Country Traversed by It.

to generate electricity for operating air compressors, derricks, rock crushers, concrete mixers, and other construction machinery.

Aside from the wagon roads already mentioned (which on the completion of the work will be resurfaced and maintained by the city for the accommodation of visitors to the valley), the first permanent construction will be the Hetch Hetchy dam and the initial unit of the city's hydro-electric development, including 20 miles of aqueduct from the Early intake to Moccasin Creek, costing about $9,000,000.

The height of the Hetch Hetchy dam will be 300 ft.; its crest length about 800 ft. The structure will be of cyclopean concrete, will have a roadway along the top, and be artistically finished to harmonize with the surrounding landscape.

The Hetch Hetchy dam to a height of 300 ft. will impound 112,000,000,000 gals. From an economic standpoint, it seems desirable to construct the Hetch Hetchy dam to its full height of 300 ft. at once instead of to a height of 200 ft. first, as provided in the Raker bill. The reason for this is that more water will thus be rendered available for the immediate generation and uninterrupted supply of electricity, which can be sold to the irrigation districts and used by the city, thus lessening the burden of the taxpayers.

To prevent contamination of the water in the reservoir, it will be necessary to clear all the trees and shrubbery from the area that will be submerged.

Of the total reservoir area of 190 acres, only 300 acres will have to be cleared in this manner, as the remainder is barren. Some of the larger trees can be converted into lumber and used for construction purposes. The remainder will be cut down and removed from the site. While this work is in progress at the dam site, work will also be started on the conduit at the Early intake; also on the main aqueduct and on a permanent power plant at Moccasin Creek.

The Early intake will be located on the Tuolumne River near the mouth of Cherry Creek, 12 miles below the Hetch Hetchy dam. It is proposed at first to use the river channel to convey water from the Hetch Hetchy reservoir to the Early intake, in order to lessen the first cost and bring into early use a 50,000-HP electric plant, the revenue from which will carry the financial burden of this portion of the outlay.

Water will be delivered from the main river channel at the Early intake by means of a temporary overflow dam 500 ft. in length and about 15 ft. high, and diverted into a pressure tunnel 10 ft. in diameter leading to Moccasin Creek, 20 miles distant.

There is only one power drop in the aqueduct line first to be constructed. The power house site for this is situated very close beside the main highway at Moccasin Creek, 6 miles easterly from where the aqueduct crosses the main Tuolumne River, and about 111 miles distant from the city hall of San Francisco, along the aqueduct line, or 150 miles around the head of the bay.

The aqueduct capacity from the Early intake down to the power house will be somewhat above 620 cu. ft. per second, which is the equivalent to 400,000,000 gals. daily.

The elevation of tail water of this power house is planned to be 800 ft. above mean sea level. The water level at the intake will be 2,315 ft. and the gross fall thus 1,125 ft.

The conduit from Moccasin Creek power house across the San Joaquin valley, a distance of 15 miles, will next be undertaken. The size of pipe for this purpose, and whether one large or two smaller ones are preferable, are economic and engineering problems which have not been definitely decided, and which demand the most careful consideration.

All of the tunnels along the aqueduct will be lined with concrete and smoothly finished, and all steel pipes will be plastered on the inside with cement mortar. The cement lining will preserve the steel pipes from rust, thus maintaining their carrying capacity undiminished.

It will be desirable to unite the westerly end of the aqueduct with the reservoir of the Spring Valley Water Co., if the same can be acquired by the city. The object is to arrange an economic plan of progressive development when practicable in order to avoid premature and unnecessary outlays for duplication of structures which would be an undue burden on the taxpayers. The completion of the Calaveras dam, now in course of construction, should add about 20,000,000 gals. per day to the present Spring Valley water supply, and provide ample water until the Hetch Hetchy system is completed.

This, however, will meet the necessary demand for a short time only, barely sufficient to permit first delivery of water from Hetch Hetchy to San Francisco.

In the future, when the demands of San Francisco warrant, the Lake Eleanor and Cherry Creek units and other higher reservoirs in the city's 417,000-acre watershed will be added to the system.

The Self-Purification and Care of Water Stored for Public Supply Purposes.

The advantages of the reservoir storage of water are many and the disadvantages are few. Formerly the expression "stagnant water" carried with it many ill-defined fears. It is now well known that still water rather than running water purifies itself the better. The greater the velocity of a stream the sooner is its pollution delivered to the consumer. Much more depends, therefore, upon the time of flow than upon the distance traversed between the point at which a stream is polluted and the point at which withdrawal of water is made. The following discussion of the self-purification of stored waters and the care of such waters when intended for public use is taken from an address by Dr. Wm. P. Mason, professor of chemistry at the Rensselaer Polytechnic Institute, before the Franklin Institute, as published in the Journal of the Institute for April, 1914. Under the heading of the care of stored waters the use of copper sulphate for the removal of algal growths is discussed and data on the amount of the material needed and the amount which can be safely used are given. The portion of Dr. Mason's paper which relates to the patrol of watersheds is omitted from this abstract.

Stagnation has its disadvantages, of course. Increase in color naturally follows if water be permitted long to remain in contact with a muddy bottom loaded with soluble extractive matters. Not only is damage to the water's physical appearance a result of such contact, but the material passing into solution is likely to furnish abundant food for those minute forms of life which carry objectionable tastes and smells to many public waters.

One condition of "standing water," namely, that of being overstocked with vegetable growths, is practically the only one toward which objection can point when considering the pros and cons of reservoir storage.

The word "vegetable" is here to be taken in its broad sense, as it should include not only those growths which would be recognized by the public at large, but also those of the minute world as well, which latter constitute one branch of that lake life known as "plankton."

As an instance of excessive overgrowth, a small lake could be named which is so loaded with dense vegetation that decay gets ahead of new growth, and the use of its water is productive of temporary diarrhœa. It is a stained water, but its color is not to be considered as a measure of its objectionable qualities for table use, as many waters of much darker tint are of excellent quality for such purpose. Color and fitness for drinking bear no relation to each other.

It is true that colorless waters are now demanded by the people, and "meadow teas" are growing in disfavor, but that change in public opinion is of recent date and is not based upon established hygienic grounds.

The deepening in color of the lower layers of a water stored upon an unclean bottom; the encouragement of growths of small organisms producing taste and smell by reason of an accumulation of extracted food suitable for their development; and a deficiency in dissolved oxygen in the bottom levels, constitute the sum of objections that can be raised to the impounding of water, and they are much more than balanced by the advantages that accrue from such storage.

The pollution from shore drainage that reaches a reservoir water is likely to be naturally much smaller in quantity than that received by a flowing stream; nevertheless, very serious pollution may occur in concentrated form, even when the efforts of the caretaker are the most earnest. Thus, the writer has seen numerous loads of stable manure spread upon the very steep banks of a small, cup-shaped distributing reservoir. The intention in this case was good, and much pride was taken in the fine lawns of the waterworks park, but a better choice of fertilizer could have been made for such a location. It is entirely possible to protect a reservoir, and to a great degree the water-shed also, if careful policing of the district be established.

When meadow lands are flooded, the extraction of food for plankton growth must of necessity take place, and we are forced to choose between stripping the proposed bottom or else depending upon the employment of some means of killing or removing the organisms that are likely to develop.

As to what could be done to rid a water of the odors due to algal and other growths, the outlook was not very hopeful previous to the appearance of the "copper sulphate process" proposed by Moore and Kellerman in 1904.

The method of applying this chemical is simple enough, and its use is very efficacious. Bought in bulk, it can be had at about 5 cts. per pound, and its distribution is readily secured by filling it into perforated buckets, or even bags, and towing the same by rowboat or launch over the reservoir surface.

(In discussing the "coppering" of reservoirs Dr. Mason has suggested that a raft carrying the bucket of copper sulphate crystals be towed behind the boat which is rowed around the reservoir. The bucket is, of course, perforated and comes in contact with the water through a hole in the raft into which it is set to a suitable depth to insure stability. By this means a careless or indifferent workman is less likely to spill the contents of the bucket at one spot and consider his work finished.—Editors.)

Decided objection was raised against such a process of "disinfecting" a public water supply, and the opposition was especially marked in England, but the use of it is still with us and is likely to stay, for the reason that the "dose" is minute and is only occasionally required, that it is reliable in results, and that experiment has shown that it is not followed by the evil consequences predicted.

It must be remembered that it is not added to the water continually but is used only at stated and widely separated intervals, namely, at those times when the "crop" of minute organisms has become so well grown as to produce objectionable effect upon the water.

Perhaps one reason why the "coppering" of reservoirs has led to so much criticism is because of the dead fish that are to be seen after the chemical has been applied. When considering this effect upon fish life, one should bear in mind that the "dose" has of necessity to be applied uniformly over the surface of the water, and each acre of such surface presumably receives the same amount, irrespective of the depth of water that the acre covers. As a result, the shallow parts of the lake receive temporarily a greater quantity of the sulphate, per cubic foot of water, than do those which are deeper; again, the entire quantity of chemical intended for the whole body of the lake is delivered to a few inches of its surface layer; therefore, until diffusion has taken place, fish which chance to swim into such water receive a very concentrated dose and are likely to be affected by it. Distribution is complete by the time the water reaches the public mains, and, moreover, the minute dose used has been more or less completely disposed of through its action upon the organisms for whose destruction it has been employed.

In a paper before the Section on Hygiene of the Eighth International Congress of Applied Chemistry, 1912, Kellerman presented the data given in Table 1 on the required amount of copper sulphates.

TABLE 1.—QUANTITY OF COPPER SULPHATE REQUIRED TO KILL VARIOUS FORMS OF ODOR-PRODUCING ORGANISMS.

Copper Sulphate Required, Expressed as Parts per Million Parts of Water.

Anabæna	.09
Asterionella	.1
Beggiatoa	5.
Chara	.2 to 5.
Cladophora	1.
Cladothrix	.2
Clathrocystis	.1
Cœlosphærium	.3
Conferva	.4 to 2.
Euglena	1.
Fragilaria	.25
Hydrodictyon	.1
Kirchneriella	5. to 10.
Leptomitus	.4
Microspora	.1
Navicula	.07
Oscillatoria	1. to .4
Peridinium	2.
Scenedesmus	5. to 10.
Spirogyra	.05 to .3
Ulothrix	.2
Uroglena	.05
Volvox	.25
Zygnema	.7

He adds a list of twelve genera of algæ that in his experience are causing trouble in reservoirs and ponds:

Number of Observed Cases.

Anabæna	27	Conferva	56
Asterionella	9	Crenothrix	13
Beggiatoa	20	Fragilaria	19
Chara	26	Navicula	21
Cladophora	17	Oscillatoria	49
Clathrocystis	23	Spirogyra	43

Jackson claims that blue-green algæ will die if the water be "coppered" one part to five million. His dose for Mellosira or Synedra is one to two million, and he claims that the former gives no odor of growth, but only that of decay. He finds that coppering runs out certain forms of organisms and substitutes others by a sort of selective action, but those thus substituted are not likely to be odor-producers; and he further notes that, while "bottom" or decomposition odors are easily shaken out by aëration, "top" odors, viz., those of growth, have to be removed by filtering out the organism, or killing them by copper sulphate, or both. In his opinion, filtration of either type is effective for removal of odors of growth, but he believes that aeration would be worse than useless for living plankton, for the reason that the agitation would tend mechanically to release the oil causing the taste, which oil is not very easily oxidized.

Naturally the cost of treatment with sulphate of copper will depend in part upon the amount of the chemical that is to be used, which in turn is determined by the kind of organism that it is intended to kill; but it may be said that a mixed growth of Mellosira and Asterionella was removed from the Troy reservoir at an expenditure of 11.9 cts. per 1,000,000 parts of water treated, labor included. The dose was one part of copper sulphate to 3,500,000 parts of water by weight.

In the article by Kellerman above quoted there are figures given indicating the safe limit for treating water with copper sulphate when certain fish are to be protected.

Copper Sulphate, Expressed as Parts per Million Parts of Water.

Black bass	2.1	Perch	.75
Carp	.3	Pickerel	.4
Catfish	.4	Suckers	.3
Goldfish	.5	Sunfish	1.2
		Trout	.14

It must be noted that these figures assume a thorough mixing of the sulphate solution with the whole body of water. They would not hold for the unequal distribution and resulting local concentrations already mentioned.

In some reservoirs which have been formed by the extensive flooding of swamp bottoms there may develop objectionable growths of crenothrix, a general term denoting an aquatic plant which at times gives much trouble because of its tendency to develop in the street mains and clog the pipes. It is often discovered quite unexpectedly, being dislodged by the current attending hydrant flushing or by the draft caused by fire engines. Dead ends are spots likely to harbor it, and its long rusty filaments have been mistaken for horse manure.

There are three types of the growth, each possessing the peculiarity of precipitating from the water in which it grows its own particular metallic hydroxide. By far the commonest of the three is Crenothrix Kuhniana, which demands iron for its development and which deposits large amounts of iron hydroxide as the result of its growth. The iron required for growth must be in solution, and the quantity demanded would seem to be about 0.3 part of iron per million.

In order that the iron may be in solution, we naturally would expect the dissolved oxygen to be low and the quantity of reducing agents, such as organic materials, to be high, and those are the conditions that we find in practice to be favorable to the development of the plant.

It is likely to be encountered in waters from swampy, peaty sources where dissolved oxygen is scanty and where the necessary iron in solution may be had. Driven wells in such localities frequently furnish it. Darkness favors its growth, and its development in city water-mains is often excessive, resulting in a material reduction of the carrying capacity of the pipes. The writer has some doubt about the "manganese" variety of Crenothrix being as rare as some think it is, he having found large quantities of manganese in a heavy Wisconsin growth. Beythien and others have, moreover, noted that the presence of manganese in water directly favors the growth of the ordinary form of Crenothrix.

Beyond the mechanical stopping of street-pipes, Crenothrix is exceedingly objectionable to the laundry interests of the community, for the reason that its rusty filaments cause "iron stains" to appear upon white linen.

Removal of the iron by oxidation and filtration is the best guard against troubles due to Crenothrix.

It must not be sweepingly assumed that all the "plankton" life is to be rated as uniformly objectionable; quite the contrary, as a reasonable degree of it acts as a distinct help in maintaining the safety of natural waters. Thus we find "bacteria eaters," such as many kinds of ciliated infusoria, rotifiers, dophnia and the like, feeding upon minute germ life, and doing so to our great advantage.

RESERVOIR STRIPPING.

With a view to avoid the troubles arising from the undue growth of odor producing organisms the stripping of the bottom and sides of reservoirs below the water line and the removal of a portion of the upper soil has been advocated and carried into practice. This, of course, entails very great expense when the surface to be stripped is at all extensive. At Columbus, Ohio, such work cost $159 per acre. In view of the expense of such treatment for large reservoirs the question, "Does it pay?" is pertinent.

For the sake of general appearances, if for no other reason, trees, shrubs and bushes should be removed. Dead, standing timber and fallen logs are most unsightly and are very likely to produce complaint from the visiting public. In other words, the reservoir site should be cleared and grubbed, with, of course, entire removal of every vestige of human habitation; but beyond that it scarcely pays to go. The portion of the flooded land lying between high-water and low-water marks should receive especial attention, for the reason that during the periods of its exposure it is capable, if uncared for, of presenting an unpleasant appearance and provoking adverse criticism; with the further objection that heavy weed growth may develop if it be long uncovered, which growth will contribute toward the production of taste and smell when the water again covers it.

In an instance where it was proposed to restore a dam that had been out of repair for over 50 years, the writer advocated the cutting off of the standing dead timber at the existing water level before closing the breach, in order to insure a better looking sheet of water when the reservoir filled. This was for appearances only, as all extractive matter had been leached out of the old vegetation long before.

Aeration, filtration, and the judicious, occasional use of copper sulphate constitute the processes at our disposal for combating the annoyance arising from algal growths, and their use will give greater satisfaction than the expensive stripping of reservoir bottoms, a treatment which was so frequently advocated in the past.

PURIFICATION IN STORAGE.

Dr. A. C. Houston, of the London Metropolitan Water Board, has undertaken some very extended researches upon the question of water purification as a result of storage. He found that in stored Thames water the death of typhoid bacteria took place rapidly, although the rate varied with the temperature of the water. In cold water they lived longer than in warm, and 50° F. seemed to be a critical point above which their mortality rate was much increased.

In his 7th Research Report, Houston states that typhoid bacilli lived in stored raw Thames water for the following lengths of time: At 32° F., five weeks; at 41° F., four weeks; at 50° F., three weeks; at 64° F., two weeks.

Even these figures do not tell the entire story. Put in more detail they read:

	At start.	One week.	Two weeks.	Three wks.	Four wks.	Five wks.
32° F. ..	103,328	47,766	980	65	34	3
41° F. ..	103,328	14,894	26	6	3	..
50° F. ..	103,328	69	14	3	..	..
64° F. ..	103,328	39	3	..	..	..

He concludes: "It is difficult to escape the belief that thirty days' storage of river water is tantamount to sterilization, so far as the microbes associated with water-borne epidemic disease are concerned."

Dr. Houston adds: "I am well satisfied that a well-stored, rapidly filtered water is likely to be safer than an unstored, slowly filtered water."

Although Dr. Houston is doubtless sound in his judgment that a great measure of safety will result from four weeks' reservoir storage of a polluted water, yet we must be assured that the period of storage is real and not simply apparent; or, in other words, we must know that all of the water really does remain in the reservoir for the specified length of time before it is used for public consumption.

Where the inlet and outlet of a reservoir are near together, as is not uncommonly the case, it makes but little difference what the capacity of the total storage may be; the water simply slips in and out again with practically as little stay as though the reservoir was a standpipe.

If the lake be long, narrow and deep, and all of its water be obliged to traverse its entire length before being taken for supply, then the conditions would appear ideal for purification of the inflowing water before the outlet was reached, and yet even under those excellent conditions it is possible to have introduced unexpected and upsetting factors, unless the watershed is regularly patrolled and inspected.

Use of Cast Lead in Making Temporary Repairs to Triplex Pump, Cracked by Freezing, at Sydney, Nova Scotia, Water Works.

Contributed by James Menzie, Superintendent of Water Works, Sydney, Nova Scotia.

A Gould's triplex pump, installed to lift water 110 ft. into a 6,000 gal. tank to supply residences in Sydney, Nova Scotia, at an elevation above that served by the city pressure was, through the carelessness of an attendant, exposed to frost and the pump and discharge main froze up. The pump casing split across the front of the well on the discharge side and the crack extended across the base of the pump and also across the suction side. Quick repairs were necessary since the pump furnished the only supply of water available to the residences previously mentioned. Temporary repairs were made by means of cast lead as follows:

The metal of the pump casing was scraped clean for about ½ in. on each side of the crack for its full length. A band of tin was then soldered along the crack. Clamps were put on to prevent the lengthening of the crack and the pump was replaced in position, but the anchor bolts were not drawn tight.

A mold was then made and so set up around the pump base that molten lead could be poured inside it. The poured lead filled the ⅜-in. space between the bottom of the pump and the pump bed, and extended around the ends and side of the pump the full height of the well. Before pouring the lead the drip cocks were replaced by nipples long enough to extend through the lead which was poured around that portion of the pump to a thickness of 2 ins.

After the cast lead cooled the mold was removed and a caulking iron was run around the edges of the lead and the pump was tightened on its base by means of the anchor bolts. As an additonal precaution plates were placed on both ends of the pump and were drawn together by bolts, thereby holding the lead in place. Finally hot pitch was applied around the edges of the lead.

When the pump was started up, following the making of the repairs here described, the leakage was very small. The repairs were intended for temporary use during the two weeks necessary in which to get a new part from the factory to replace the one cracked. The new part, however, has not been needed and it does not appear likely that it will be needed.

The cost of making the repairs was $11.95, being made up of $9.25 for labor and $2.70 for material.

Bridges

Design Features of the Williams Reinforced Concrete Highway Bridge Over the Scioto River, Columbus, Ohio.

(Staff Article.)

The Williams Bridge, which is under construction across the Scioto River in Columbus, Ohio, is a reinforced concrete highway structure having a total length of 1,263 ft. 3 ins., out to out, and an over all width of 30 ft. 4 ins. It has a 24-ft. clear roadway and two 2-ft. clear sidewalks, and is provided with an ornamental concrete railing which extends 3 ft. 7 ins. above the sidewalk surface. Starting at the east end of the bridge, at the center of the retaining wall pilaster, the composition of the structure is as follows: 177 ft. 3 ins. of concrete retaining wall construction; 212 ft. 9 ins. of reinforced concrete trestle, comprising six bents and consisting of a beam-and-slab roadway surface supported on reinforced concrete columns; a series of six reinforced concrete arch spans having lengths, center to center of abutments, of 70 ft. 8 ins.; 80 ft. 7 ins., 91 ft. 6 ins., 102 ft. 5 ins., 115 ft. 4 ins., and 209 ft. 0 ins.; 176 ft. 0 ins. of reinforced concrete trestle, consisting of five bents; and 24 ft. 3 ins. of concrete retaining wall construction at the west end of the bridge. The bridge is on a tangent from the east end at station 2 + 00 to station 13 + 32, at which point it deflects to the north at an angle of 9° 2′. The structure is on a 0.68 per cent upgrade from the east end of the bridge to the west end. The arch spans are of the ribbed type, with spandrel columns, cross girders, longitudinal beams and a reinforced concrete deck. The roadway surface is paved with brick. Figure 1 shows an outline drawing of the bridge, with elevations and general dimensions.

LOADS AND ALLOWABLE STRESSES.

In determining the dead load for computing stresses the weight of the 4-in. brick surface was taken at 40 lbs. per square foot; the 3-in. sand cushion, at 30 lbs. per square foot; and the concrete, at 144 lbs. per cubic foot.

The concentrated live load used was a 24-ton truck, with 40 per cent impact, the assumed wheel base being 5x10 ft. A uniform live load of 100 lbs. per square foot over the remainder of the roadway and over the sidewalk was taken.

The allowable tension in the reinforcing steel was 16,000 lbs. per square inch, and the allowable compression in the concrete due to bending was 700 lbs. per square inch.

All footings are founded on rock, a maximum allowable pressure of 7 tons per square foot under unsymmetrical loads being used.

MATERIALS AND SPECIFICATIONS.

The following extracts from the specifications indicate the character of various parts of the work:

Concrete Work.—Four classes of concrete will be used in the bridge. Class A consists of 1:2½:5 concrete with embedded stones, the stones to be placed not nearer than 6 ins. to each other, nor nearer than 9 ins. to any exposed face. This concrete is to be used in the footings of abutments and piers, except in the footings for piers Nos. 9 and 12. Class B consists of 1:2:4 concrete, and it is to be used in the retaining walls above the footings, in the solid portions of arches and piers, in the retaining wall at pier No. 12 and in retaining wall at pier No. 11. Class C consists of 1:2:4 concrete, and it is to be used for spandrel posts, trestle posts, hollow portions of arch piers, sidewalks, deck or roadway construction and pier struts. Class D consists of 1:2:4 concrete, and it is used in arch ribs and arch struts, or horizontal braces.

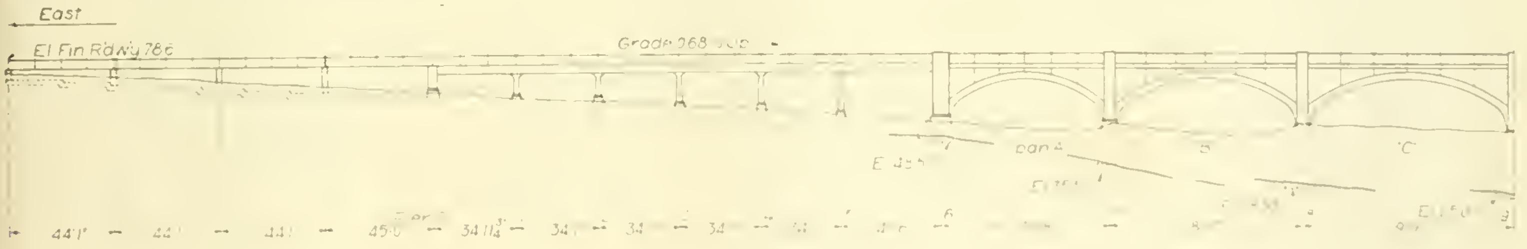
Cast stone shall be used for ornamental work, railings, posts, lamp posts and standards. In the construction of the railing, the base, the cap and all posts, the concrete shall be cast in place and given a rubbed finish, except when bush-hammered surfaces are specified. The webs shall be cast in units and removed from the molds while green, then scrubbed with brushes to remove the skin from same and expose the aggregate. All cast stone shall be made of the proportions of 1 part Portland cement, 1 to 1½ parts Canadian sand, 1 part limestone (¼ to ½ ins. in size), and 1 to 1½ parts of silica gravel (¼ to ⅜ in. in size). The engineer is to determine the exact proportions from sample blocks to be made by the contractor at his expense.

All concrete proportions are based on the assumption that 1 bbl. of Portland cement is equivalent to 3.8 cu. ft. The quantities of materials required for 1 cu. yd. of concrete of various proportions are given in the accompanying table.

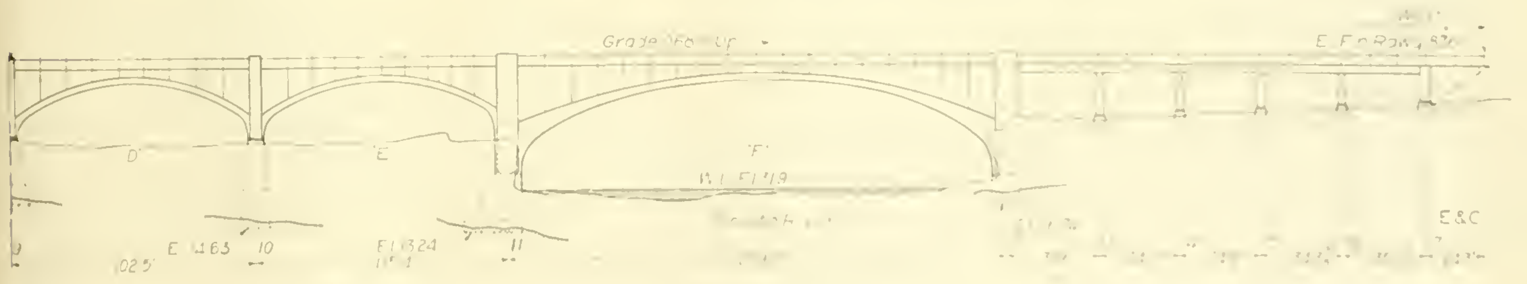
Proportion	Cement, bbls.	Sand, cu. yds.	Stone or gravel, cu. yds.
1:1½:3.	2.00	0.42	0.84
1:2:4.	1.57	0.44	0.88
1:2½:5. . . .	1.30	0.46	0.92

Reinforcing Steel.—Vertical stirrups shall always pass under the main tension bars and be securely wired thereto. All reinforcing bars shall be medium, open hearth steel, and shall be wired together at every intersection. Splices shall be at least 10 diameters in length and shall be securely wired in at least two places, or as the engineer may direct.

Forms and Centers.—Forms shall in general be composed of 2-in. matched sheeting, and shall be surfaced on the side next to the concrete for all surface which will be exposed



(EAST PORTION.)



(WEST PORTION)

Fig. 1. Outline Drawing of Williams Bridge, Columbus, Ohio, Showing Type of Structure and General Dimensions.

in the finished work. In general forms shall not be removed from concrete in less than 14 days in summer and 28 days in winter, except for vertical surfaces, etc., of numbers which carry no loads, the forms of which may be removed in from 3 to 6 days. Centers shall not be struck from arches in less than 28 days, and in no case without first notifying the engineer in charge.

In proportioning forms and centering, the concrete shall be treated as a liquid of its full weight for vertical loads and one-half its weight for horizontal pressures. In constructing timber centers for arches an allowance for settlement shall be made equal to ¾ in. for each 30 ft. of vertical height.

Surface Finish of Concrete.—The exposed surfaces of arches, piers, spandrel walls, railings, beams, retaining walls, abutments, columns, bases, etc., shall be finished by rubbing in a thin mortar with carborundum brick, sufficient to fill all pores and cavities and to remove all board marks. Where indicated on the drawings the panels shall be bush hammered.

Waterproofing.—The top of the concrete roadway slab and the inside surfaces of retaining walls shall be waterproofed with a heavy, impervious coating of hot waterproofing pitch, with two-ply elastic fabric, 2 ft. in width, over all expansion joints. The elastic fabric shall be "Tartar" or its equal, applied by first swabbing the concrete with hot pitch and then applying two alternate layers of fabric and pitch.

DESIGN FEATURES.

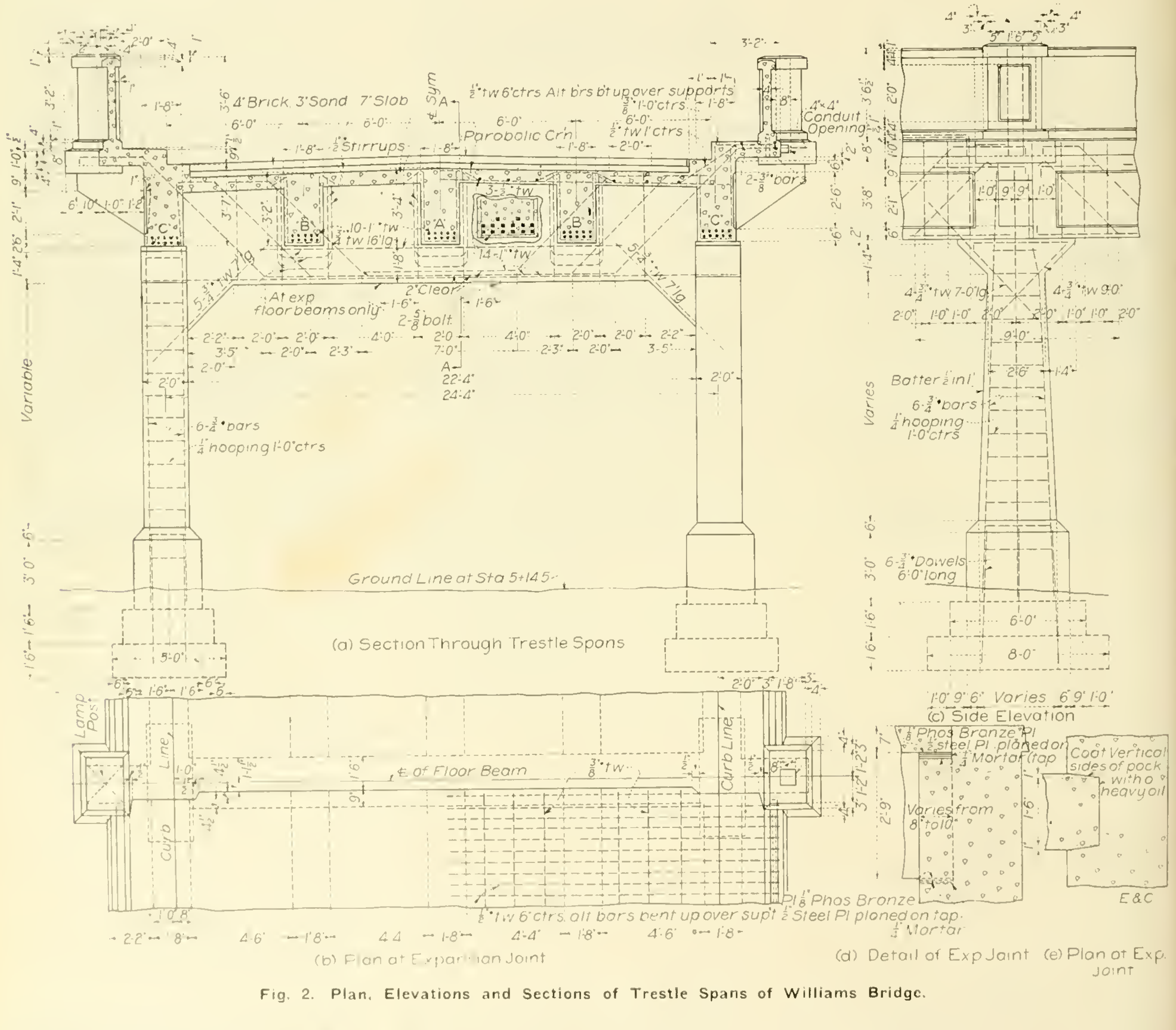
Retaining Wall Approaches.—The east retaining wall approach has a length of 177 ft. 9 ins. from center of end pilaster to center of the abutment which marks the beginning of the reinforced concrete trestle (see Fig. 1). On account of the slope of the ground the footings are stepped. The side panels of the retaining walls are bush-hammered, and the walls are surmounted with ornamental concrete railings.

The west approach of the bridge consists of an earth fill and a 24-ft. 3-in. panel of retaining wall construction adjoining the reinforced concrete trestle spans.

Reinforced Concrete Trestle.—The portion of the bridge between the east retaining wall approach and the arch spans has a length of 212 ft. 9 ins., and consists of six reinforced concrete beam-and-slab spans on reinforced concrete columns. The lengths of these spans, center to center of columns, starting with the east span, are: one span at 31 ft. 11¾ ins.; four spans at 34 ft. 1 in.; and one span at 11 ft. 6 ins. These spans are between piers Nos. 0 and 6 (see Fig. 1). The roadway surface is brick, laid on a 3-in. cushion of sand. The floor construction consists of a reinforced concrete slab, longitudinal beams, and cross girders. The latter frame into, and are carried by, reinforced concrete columns on spread footings. Figure 2 shows detail drawings of the principal features of the trestle spans. Figure 2 (a) gives a cross-section of a typical trestle span, and shows the type of construction, the principal dimensions and the reinforcement; Fig. 2 (b) shows a part plan of a span adjacent to an expansion joint, and gives the principal dimensions and the arrangement and size of the reinforcement in the floor slab; Fig. 2 (c) shows a side elevation of the columns, footing and a part of the superstructure; and Figs. 2 (d) and (e) show details of the expansion joints. A feature of this joint is the use of phosphor-bronze expansion plate.

The portion of the bridge between the west retaining wall approach and the arch spans has a length of 176 ft. 0⅛ in., and consists of five reinforced concrete spans of similar construction to those just described. The length of these spans, center to center of columns, beginning with the west span are: 38 ft. 6 ins.; 34 ft. 0 in.; 31 ft. 0 in.; 31 ft. 3⅝ ins., and 35 ft. 2½ ins. These spans are between piers Nos. 17 and 12 (see Fig. 1).

Arch Spans.—Between piers Nos. 6 and 12 (see Fig. 1) the bridge consists of six reinforced concrete arch spans of varying lengths. The lengths of these spans, center to center of piers, are: Span "A," 70 ft. 8 ins.; "B," 80 ft. 7 ins.; "C," 91 ft. 6 ins.; "D," 102 ft. 5 ins.; "E," 115 ft. 1 ins.; and "F," 209 ft. 0 in. The rises of the spans are: Span "A," 12 ft. 0 in.; "B," 13 ft. 9 ins.; "C," 15 ft. 9¾ ins.; "D,"



(a) Section Through Trestle Spans

(b) Plan at Expansion Joint

(c) Side Elevation

(d) Detail of Exp Joint (e) Plan of Exp. Joint

Fig. 2. Plan, Elevations and Sections of Trestle Spans of Williams Bridge.

18 ft. 2 3/16 ins.; "E," 20 ft. 10¾ ins.; and "F," 36 ft. 4 3/16 ins. These dimensions do not consider the rounded portion near the spring lines of the arches. The piers rest on solid rock.

Figure 3 shows elevations and sections of span F, the longest span. The arch consists of two rectangular ribs, spaced 21 ft. apart on centers and connected by five intermediate reinforced concrete struts. The cross-section of each rib at the crown is 48x63 ins. and at the spring line, 48x96 ins. The spandrel columns support reinforced concrete cross girders, into which frame the longitudinal concrete beams which carry the 7-in. reinforced concrete floor slab. The divided piers are braced at points near the spring lines of the arch ribs. Figure 3 (a) shows an elevation of this arch and its piers; Fig. 3 (b) shows a horizontal section through pier No. 11 and the spandrel columns; Fig. 3 (c) shows a section through the arch ribs and gives details of the strut which connects the arch ribs; Fig. 3 (d) shows another section of the arch rib; Fig. 3 (e) shows a section through pier No. 11 and also shows the footing plan; Fig. 3 (f) shows a section through pier No. 12 and also shows a plan of the small wing wall; and Fig. 3 (g) shows a section of the wing wall.

Figure 4 (p. 646) gives some floor details of the arch spans. Figure 4 (a) shows a half cross-section of the roadway near the spring line of the arches; Fig. 4 (b) shows a half cross-section of the roadway at the crown of the arch; Fig. 4 (c shows a plan of a part of the roadway slab; Fig. 4 (d) shows a cross-section of the roadway slab; and Fig. 4 (e) shows a longitudinal section through a part of the floor of arch F, the construction of the floors for the other arches being similar. The cross-sectional areas at the crown of the arch ribs for the various spans are shown in Fig. 4 (b). The cross-sectional areas at the spring line are: "A," 30x40 ins., "B," 30x43 ins., "C," 30x46 ins.; "D," 30x50 ins., "E," 30x54 ins., and "F," 48x96 ins.

Figure 5 (p. 646) gives elevations and sections of the upper part of pier No. 12, and illustrates the type of construction used. Figure 5 (a) shows an end elevation, Fig. 5 (b) shows a cross-section of the roadway near pier No. 12 and an end elevation of the pier, and Fig. 5 (c) shows a horizontal section of the pier.

QUANTITIES OF MATERIALS AND UNIT PRICES

The estimated quantities of materials and the unit prices bid for this bridge are given in Table 1, the contract price of the structure being $96,972.

PERSONNEL

Those connected with the work described are Richard Sinclair, Lewis Davis and Thos. F. Humphreys, county commissioners; John Peake, county surveyor; Walter Braun, county bridge engineer; and Wilbur J. Watson, Cleveland, consulting engineer. The contractor for the work is D. W. McGrath of Columbus, Ohio.

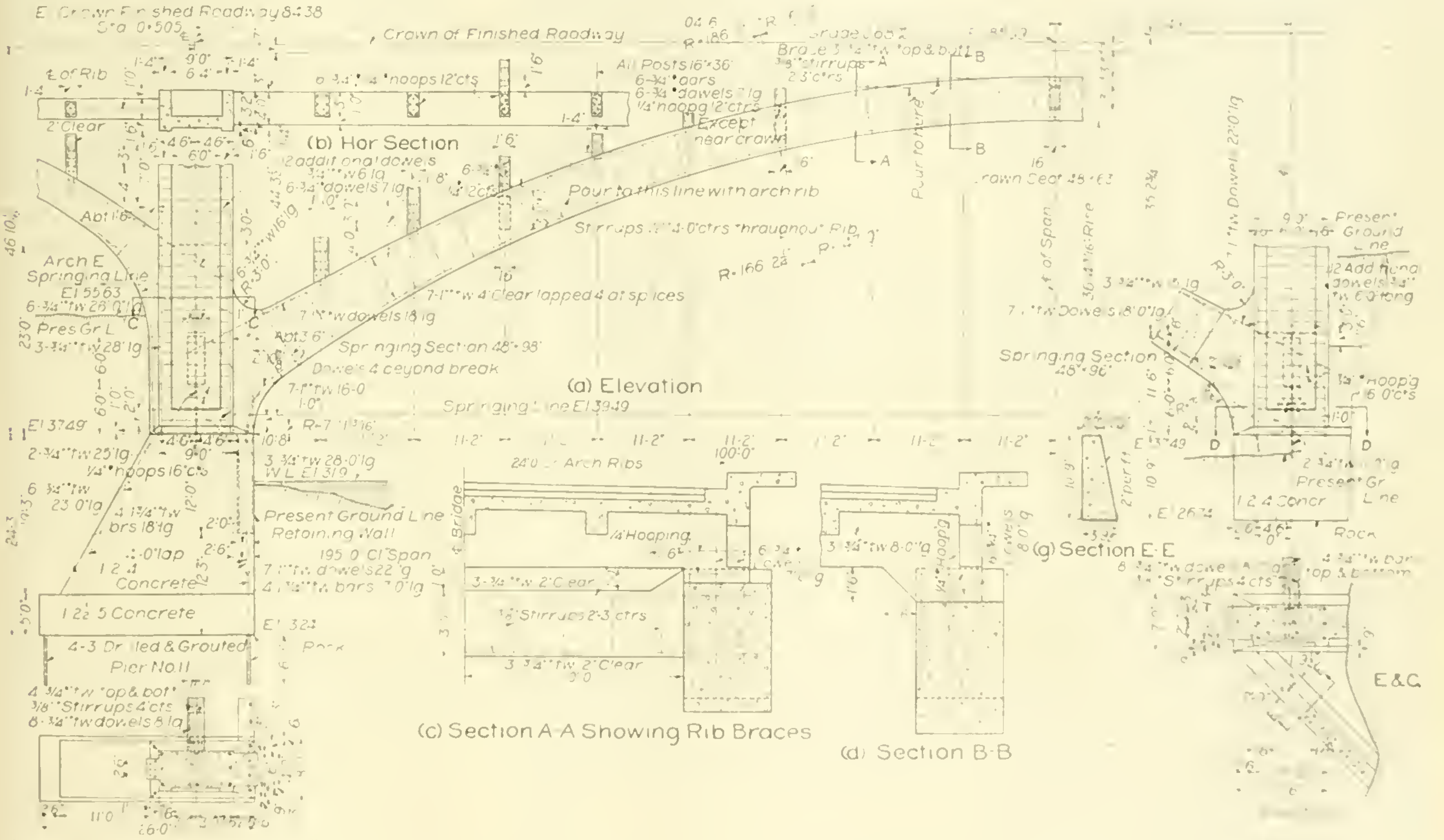


Fig. 3. Elevation and Sections of the Arch Rib and Piers of the Longest Arch Span of Williams Bridge.

TABLE 1.—ESTIMATED QUANTITIES, UNIT PRICES BID, AND TOTAL BID FOR EACH ITEM OF WILLIAMS BRIDGE.

Item and Quantity	Unit bid.	Total bid
Dry excavation and backfill, 2,300 cu. yds.	$1.50	$3,450
Wet excavation and backfill, 700 cu. yds.	2.50	1,750
Rock excavation and backfill, 700 cu. yds.	2.75	1,925
Class A concrete, 400 cu. yds	6.00	2,400
Class B concrete, 1,600 cu. yds	8.50	13,600
Class C concrete, 2,000 cu. yds	10.50	21,000
Class D concrete, 750 cu. yds	23.00	17,250
Concrete railing, 2,530 lin. ft	4.00	10,120
Reinforcing steel, 400,000 lbs	0.02¼	9,000
Curb and finish angles, 22,000 lbs	0.06	1,320
Embankment, 2,800 cu. yds	0.60	1,680
Brick paving, 2,600 sq. yds.	1.35	3,510
Lamp posts, complete, 15	20.00	300
Scuppers and 6 in. tile drains, 6	2[illegible]	[illegible]
Bush-hammered finish, 5,000 sq. ft	0.10	500
Rubbed finish, 28,000 sq. ft	0.06½	1,820
Waterproofing, 3,300 sq. yds	0.50	1,650
Sidewalk finish, 5,100 sq. ft	0.05	255
Grout-filled drilled holes, 300 lin. ft	0.50	150
Cast steel surfaced expansion plates, 1,600 lbs.	0.16	256
Rolled steel surfaced expansion plates, 5,300 lbs.	0.05	265
⅛-in. phosphor-bronze plates, 1,750 lbs.	0.40	700
Cofferdams, sheeting and pumping, complete		2,000
Conduits, wiring, fuse boxes, terminal boxes, etc., complete		1,421
Grand total, all items		$96,972

Comment on Mr. J. E. Greiner's Paper, "Rolling Loads on Bridges."

To the Editors:—A modern trend in the matter of railroad bridge designing is shown in a recent bulletin of the American Railway Engineering Association, namely, Vol. 15, No. 161. In a discussion on "Rolling Loads on Bridges," Mr. J. E. Greiner makes some recommendations which, if taken seriously, would totally demoralize railroad bridge design. Based on opinion solely, for no data nor tests whatever are quoted, Mr. Greiner makes the statement that railroad bridges can, with perfect safetly, be overloaded all the way from 50 to 100 per cent. And other engineers agree with him, on the same basis. It is a good thing that there are rules and laws which require that stability be grounded on something better than the opinions of men; there ought to be more of them, especially the laws.

In Vol. 9 of the American Railway Engineering and Maintenance of Way Association, pp. 219 and 289, Mr. Greiner gives a report of his committee on "Iron and Steel Structures," so that these deliverances may be taken as the opinion of the Association. Mr. Greiner says, "Unit strains in tension to the extent of 26,000 lbs. in structural open hearth steel and 22,000 lbs. in wrought iron will not, in themselves, be sufficient justification for suspending traffic or condemning the structures." Again he says, "That means if you cut down speeds you can run heavier overloads. It would mean practically taking a 60-ft. girder span at full speed on an overload of about 71 per cent, while at slow speed the overload would be something like 132 per cent."

In the bulletin above referred to Mr. Greiner states that "we know" from numerous tests and long experience that properly designed bridges in good condition will safely withstand an overload of 50 per cent without any traffic or speed regulation, and if speed is regulated an occasional overload of 100 per cent. No tests are cited; the experience is not described. This is stated as more conservative than has been the successful practice of a number of railway engineers. Mr. Greiner further states "An E-50 American Railway Engineering Association Specification bridge is a good and economical type with sufficient strength to safely carry, in regular unrestricted service, the heaviest locomotives that can be safely operated without a possible complete revision of present standard clearances."

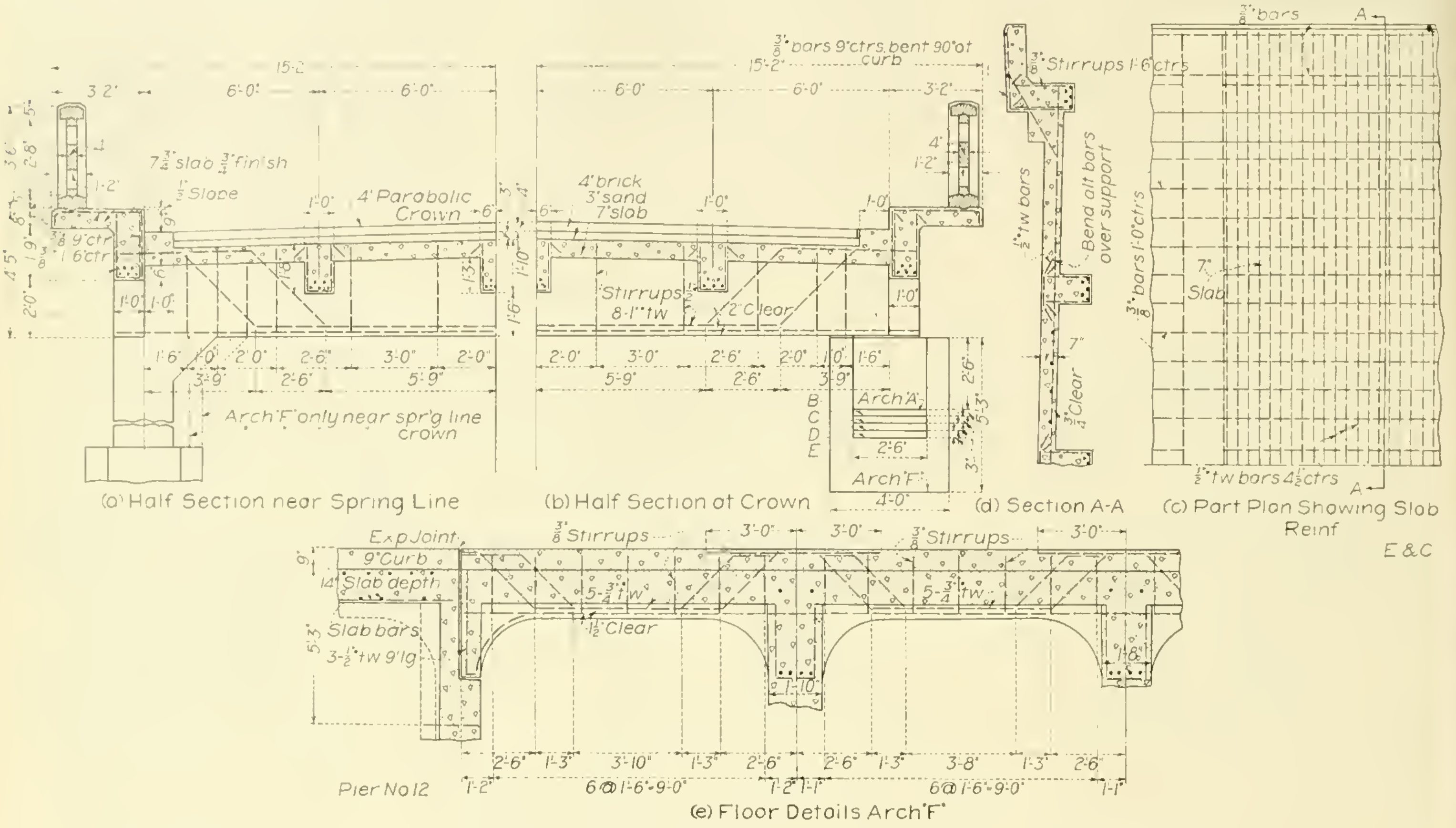

(a) Half Section near Spring Line
(b) Half Section at Crown
(d) Section A-A
(c) Part Plan Showing Slab Reinf
(e) Floor Details Arch "F"
Pier No 12
Arch "F" only near spr'g line crown
4' Parabolic Crown
E&C

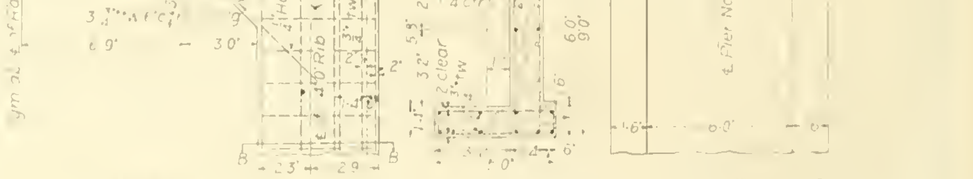
Fig. 4. Plan and Sections of Floor Construction for Arch Spans of Williams Bridge.

Experience is good: there is nothing to equal it. Opinion is good, when nothing else is available. Suppose that experience consisted of watching a roof truss for 40 years, a truss that calculations showed to be weak. Such experiences would doubtless say that the truss was perfectly safe. The truss the writer has in mind stood for 42 years and then collapsed. Suppose it consisted in watching a railroad girder span for 30 years, part of which time it had been loaded somewhat beyond its calculated capacity. The experience would say that the span was perfectly safe. The span the writer has in mind was in Pittsburgh; it failed after more than 30 years of service under usual conditions, slow speed of a locomotive not very heavy. The occurrence took place May 7, 1900, and the writer was on the ground before the police shoved the inquisitive back. By his calculation the unit stress was something less than 14,000 lbs. per square inch in tension on wrought iron, a little more than half of what Mr. Greiner considers perfectly safe—perhaps 40 per cent of overload above the designer's assumption.

A short time ago another fatal wreck occurred on a viaduct that had stood for more than 20 years. A box girder, woefully bad in detail, for it was hollow where there ought to have been stiffeners over the column, crushed down over the column. If a detail so abominable as this could stand 20 years of heavy loads and constant traffic, would not experience pronounce it good?

None of these cases were of wearing out from wasting of the metal. The weakness was original and was ascertainable. Calculation and logical designing were infinitely better than decades of experience and observation in determining the sufficiency of these structures.

"Numerous tests" are also good, especially when the tests are carried to a conclusion. A contractor tests his equipment again and again, and after lifting a given load hundreds of times a chain will break under the same or less load. Calculation will show him that the chain was overloaded every time it lifted that load, but it only failed at the last time. This is in perfect agreement with laboratory tests. A large number of repetitions of loads near the elastic limit of steel will eventually break a specimen. The steel is rendered brittle by excessive loads and finally breaks. Chain users understand this and heat or anneal their chains occasionally to toughen them. Bridge users never anneal their old ill-used bridges.



(b) Half Section A-A (c) Section B-B (d) Elev Pier No 12

Fig. 5. Elevations and Sections of Pier No. 12 of Arch Span of Williams Bridge.

If the numerous tests referred to by Mr. Greiner have been published it would be well to call attention to them and show how they furnish a basis for shattering all standards for bridge design, for if 26,000 lbs. per square inch is perfectly safe, why waste money by using 16,000 lbs.? If such units constitute sound engineering in railroad bridges, where unbalanced loads go hammering over uneven tracks, to spare the railroads the expense of replacing old and weak bridges, why not use even higher units in buildings and in bridges where automobiles with their gum shoes go quietly by?

The elastic limit is the practical ultimate strength of bridge members, both in tension and in compression. This statement admits of no controversy. It is established beyond question that stresses at or near (below) the elastic limit will, under repeated application, cause failure.

The yield point of wrought iron specimen tests is usually about 26,000 lbs. per square inch. This is the point where "drop of beam" is observed. The true elastic limit is very much less, probably in the neighborhood of 20,000 lbs. The writer has taken steel with a nominal yield point of 40,000 lbs. per square inch, and by testing it under a very slow speed has found it to have an elastic limit of about 32,000 lbs. per square inch.

What degree of safety would there be in a structure whose tension members are stressed 22,000 lbs. in wrought iron having an elastic limit of 20,000 lbs.? The structure might stand it for years.

What degree of safety would there be in a structure whose tension members are stressed 24,000 to 32,000 lbs. per square inch (50 to 100 per cent overload) in steel whose elastic limit is 32,000 lbs., or perhaps less?

Compression members, by the American Railway Engineering Association Specifications, have the same units as tension members (reduced for column length). These would be subject to the same excess loading in a bridge. What do tests show in regard to compression members? The best set of tests extant, because they are made on actual-size bridge members of good average shop work, are those made by the late Mr. C. P. Buchanan and published in "Engineering News," Dec. 26, 1907. Mr. Buchanan in this article gives the elastic limit or yield point of these several tests, 19 in all. The maximum loads are somewhat higher than the yield point or point of permanent set, in some cases the difference is but a few thousand pounds, in others the ultimate strength is about double the load at permanent set. It is the yield point that counts. It is under this load or less that the member would fail under a large number of repetitions.

Nine of the tests of wrought iron compression members showed a permanent set or yield point between 13,200 and 20,500 lbs. per square inch (average 16,200 lbs. per square inch). These members, however, failed in the end details. Two steel members also failed in the details, showing permanent set at 23,600 and 24,700 lbs. per square inch respectively.

Three wrought iron members failed in the body, showing permanent sets at 17,000, 17,200, and 20,150 lbs. per square inch, respectively. Five steel members failed in the body showing permanent set at loads of from 12,600 to 25,500 lbs. per square inch (average 19,700 lbs. per square inch). The average maximum load was 31,200 lbs. per square inch. Four columns had a ratio of slenderness of 46 and less, the fifth had a ratio of 97 (ultimate strength 27,790 lbs. per square inch).

All of these were members having a ratio of slenderness between 30 and 100, except one wrought iron member whose ratio was 120 and whose permanent set was 20,500 lbs. per square inch, the highest value of all the wrought iron members.

These steel members would probably all fill every requirement of the American Railway Engineering Association Specifications. They were ordinary standard bridge members, the kind Mr. Greiner and other engineers would subject with equanimity to 50 to 100 per cent overload.

What degree of safety would there be in a structure whose compression members are stressed 21,000 to 32,000 lbs. per square inch (reduced for column length) when those members have a yield point of 19,700 lbs. and an ultimate strength of 31,200 lbs. per square inch as shown by tests?

Six tests on well-made bridge members described by J. A. L. Waddell (in Trans. Am. Soc. C. E., Vol. LXIII, p. 250) show great regularity. In three of these bridge members $\frac{l}{r}$ was 27, and the elastic limit was 28,800 lbs. per square inch—just about 100 per cent more than the safe load by the A. R. E. A. Specifications. In the other three members $\frac{l}{r}$ was 81, and the elastic limit was 16,800 lbs. per square inch—60 per cent greater than the safe load. What would happen to these members under repeated applications of loads 50 to 122 per cent greater than that allowed by the above specifications? The average ultimate load of the first three members was 39,200 lbs. and of the other three 30,500 lbs. per square inch—less than three times the A. R. E. A. safe load. The old notion about a railroad bridge *being designed* with a factor of safety of four or five is a mistaken one. If any engineers still fancy that it has any meaning it would be well for them to revise their fancies.

Now where is there any room for opinion when bald facts bellow out the truth in such unmistakable terms?

Very truly yours,
Edward Godfrey.

Monongahela Bank Bldg., Pittsburgh, Pa.,
May 2, 1914.

(A copy of the above letter was submitted to the author of the article criticised, who replied as follows:—Editors.)

To THE EDITORS:—In reply to the comments of Mr. Godfrey I shall merely call attention to the fact that the very first paragraph of his communication indicates either that he has not read my paper carefully or that he is not familiar with the subject which he endeavors to discuss.

In the first place the paper in question contains no recommendations "which if taken seriously would totally demoralize railroad bridge design." In fact all statements made are consistent with good, substantial, safe and standard railroad practice in the design and maintenance of bridges.

Mr. Godfrey says, "based on opinion solely, for no data nor tests whatever are quoted. Mr. Greiner makes the statement that railroad bridges can with perfect safety be overloaded all the way from 50 to 100 per cent." Now what I did say was as follows, and I have emphasized some essential provisional statements which Mr. Godfrey has ignored:

"All bridgemen know that *properly designed bridges* as well as steel hopper cars, may be loaded considerably beyond their nominal capacity, and that they will carry a definite amount of overload regularly and continuously without requiring any closer attention than is usually bestowed under ordinary good maintenance conditions. This capacity for overload provides to a large extent for future increases and developments."

"We know from *numerous tests and long experience* that bridges *properly designed and constructed of proper material and with members proportioned in accordance with specifications equally as good as the standard adopted by the American Railway Engineering Association, so long as maintained in good condition,* will safely withstand an overload of 50 per cent without any traffic or speed restrictions; that such a bridge may be subjected to an occasional overload considerably in excess of 50 per cent, and this without restrictions; and if the speed is regulated, the bridge will stand an occasional overload of 100 per cent."

"This statement is consistent with the writer's personal experience with the maintenance of structures in the past 25 years, and is somewhat more conservative than has been the successful practice of a number of railway engineers. *Therefore, it should be clearly understood by the operating officials of railways that a bridge of the nominal E-50 capacity, that is, one designed for Cooper's E-50 loading in accordance with the American Railway Engineering Association's Standard Specifications, will not reach its full regular traffic capacity* until the different classes of engines now in service shall have about the weights given in Table 5, and an E-60 bridge not until these engines have increased to the extent shown in Table 6."

There is nothing new to experienced bridge engineers in the above statement. This was thoroughly discussed before the American Railway Engineering Association six years ago, and the recommended standard practice as to carrying capacity of bridges adopted by this Association in 1908 is based thereon, and still prevails throughout the country. Instead of demoralizing railroad bridge design, as Mr. Godfrey says it would do if taken seriously, it has had a decided influence upon standardizing railroad bridge design, and *is taken very seriously* by railroad bridge engineers.

Mr. Godfrey says, "It is a good thing that there are rules and laws which require that stability be founded on something better than the opinions of men: there ought to be more of them, especially the laws." I heartily agree with him, but if there are to be more laws they ought to be framed by someone familiar with the subject to which they relate.

As the paper on "Rolling Loads on Bridges" is simply a report covering the heaviest locomotives of each type now in service or under consideration by the various railroads, their relative effects on E-50 and E-60 bridges, the capacity of these classes of bridges for future increase in weight of locomotives and proper specification requirements, it is essentially a presentation of facts; but there appears to be "plenty of room for opinion" as to whether present bridges have sufficient strength even "when such bald facts bellow out the truth in such unmistakable terms."

As Mr. Godfrey's remarks are founded upon an erroneous interpretation of statements made by the writer and an evident misunderstanding of the essential substance of the bridge committees' report, made to the American Railway Engineering Association in 1908, the writer has no further comments to make, except possibly a recommendation that anyone who desires to quote from or make use of the recommendations of the 1908 report and the paper on "Rolling Loads on Bridges" should first read them with sufficient care to enable him to understand clearly just what is said and meant.

Very truly yours,
J. E. Greiner.

Baltimore, Md., May 19, 1914.

Sewerage

Procedure in Designing New and Relief Storm Water Sewers in Cincinnati.

The portion of the recently issued report on a plan of sewerage for Cincinnati which relates to the design of new and relief sewers, was prepared by Mr. F. J. Van Hook. Comment upon the complete report is made in the editorial section of this issue. The present article relates wholly to the methods of analysis employed in designing the new and relief sewers.

A critical study of the existing sewerage system of Cincinnati was made with a view to recommending such improvements as would provide relief in districts subject to flooding in times of storm, and such new sewers as may be required for districts not provided with sewerage facilities. In the design for the new sewers recommended the separate system of sewerage was employed. Since the quantity of house sewage is relatively very small, as compared with the storm water flow, it is evident that the problem presented is one of storm sewer design.

RAINFALL AND RUN-OFF

Before taking up the detailed studies for sewerage improvements, analyses were made of the several factors controlling the determination of the amount of surface drainage for which provision should be made in the design of storm sewers. The factors considered are as follows: 1. Intensity of rate of rainfall. 2. Relation between rainfall and run-off, or the proportion of rainfall which immediately reaches the sewers. 3. Extent and shape of the area to be drained. 4. Surface slopes.

Many attempts have been made to establish relations between these factors and to express them in formulæ. As there appeared to be no method which had been regularly and continuously used in Cincinnati for the determination of the amount of storm drainage for which sewers and drains should provide, a review was made of the more commonly accepted methods and formulæ to ascertain if any one of these was applicable to local conditions.

TABLE I. — RATES OF PRECIPITATION WHICH FIX LOWER LIMIT OF EXCESSIVE PRECIPITATION IN STORMS SINCE 1901.

Period (minutes).	Rate (inches).	Period (minutes)	Rate (inches).
5	0.25	40	0.60
10	0.30	45	0.65
15	0.35	50	0.70
20	0.40	60	0.75
25	0.45	80	0.80
30	0.50	100	0.90
35	0.55	120	1.00

Most of the run-off formulæ are based on conditions existing in the particular city or locality for which they were derived and it is, therefore, evident that these should not be used excepting under similar conditions. Also, some formulæ are made dependent upon an adopted maximum rate of rainfall which is kept constant. On the other hand, the use of the so-called "rational" method of analysis through the introduction of the element of time permits of a varying rate of rainfall immediately dependent upon the area, shape and slope of the territory to be drained.

Also, on account of the unusual topographic features prevailing in Cincinnati, producing numerous small drainage areas with comparatively steep surface slopes in the outlying districts and flat slopes in the older portions of the city, it is important that the proper rainfall intensity and run-off for short periods of time should be accurately determined.

In view of these facts, it was concluded to adopt the "rational" method in determining the amount of storm drainage for which provision should be made. This formula may be expressed as follows:

$$Q = A\,C\,I$$

in which,

Q=the discharge in cubic feet per second.
A=the area drained in acres.
C=the ratio of run-off to rainfall.
I=the intensity or rate of rainfall during the period of concentration.

The procedure in this method of analysis is as follows:

1. The time is first approximated by observing the surface or sewer slopes, estimating the sizes of pipe necessary and, assuming that they run full, determining the velocities. The time necessary for flow from the remote portions of the drainage area under consideration can then be calculated.

2. With the time factor approximated, the intensity or rate of rainfall is determined from the rainfall intensity curve or equation.

3. The run-off co-efficient C, or reduction factor, is then determined by estimating the probable future increase in the percentage of impervious area.

4. Having determined the rate of rainfall and the run-off co-efficient, the discharge may be calculated and the sizes and grades of

TABLE III.—RUN-OFF FROM IMPROVED AREAS IN TYPICAL TENEMENT DISTRICT (CLARK STREET).

Area—60.0 acres.
Density—135 persons per acre.
Character of District—Mostly two-story dwellings and stores, with paved interior walks and yards. Essentially a tenement district.
Soil—Clayey sand overlying beds of sand and gravel.

Character of improvement.	Area in sq. ft.	Per cent of total area.	Estimated degree of imperviousness.	Equivalent imper. area sq. ft.	Per cent total area impervious.
Roofs	....	.....	...	....	38.2
Residences—Public and commercial.	981,200	37.6	.90	883,080	....
Barns and sheds	163,500	6.3	.70	114,450	....
Interior walks	....	.....	...	....	6.3
Brick	220,000	8.4	.40	88,000	....
Cement	100,400	3.8	.75	75,300	....
Street walks	....	.....	...	....	7.0
Brick	39,500	1.5	.40	15,800	....
Cement	222,600	8.5	.75	166,950	....
Street pavements	....	.....	...	....	14.4
Asphalt, brick, wood block	393,900	15.1	.85	334,815	....
Granite block	29,000	1.1	.75	21,750	....
Macadam and cobble	46,800	1.8	.40	18,720	....
Unimproved	....	.....	...	....	4.0
Unpaved yards and lawns	415,930	15.9	.25	103,982	....
Total	2,612,830	100.0	...	1,822,847	69.9

Impervious coefficient for district, 69.9%.

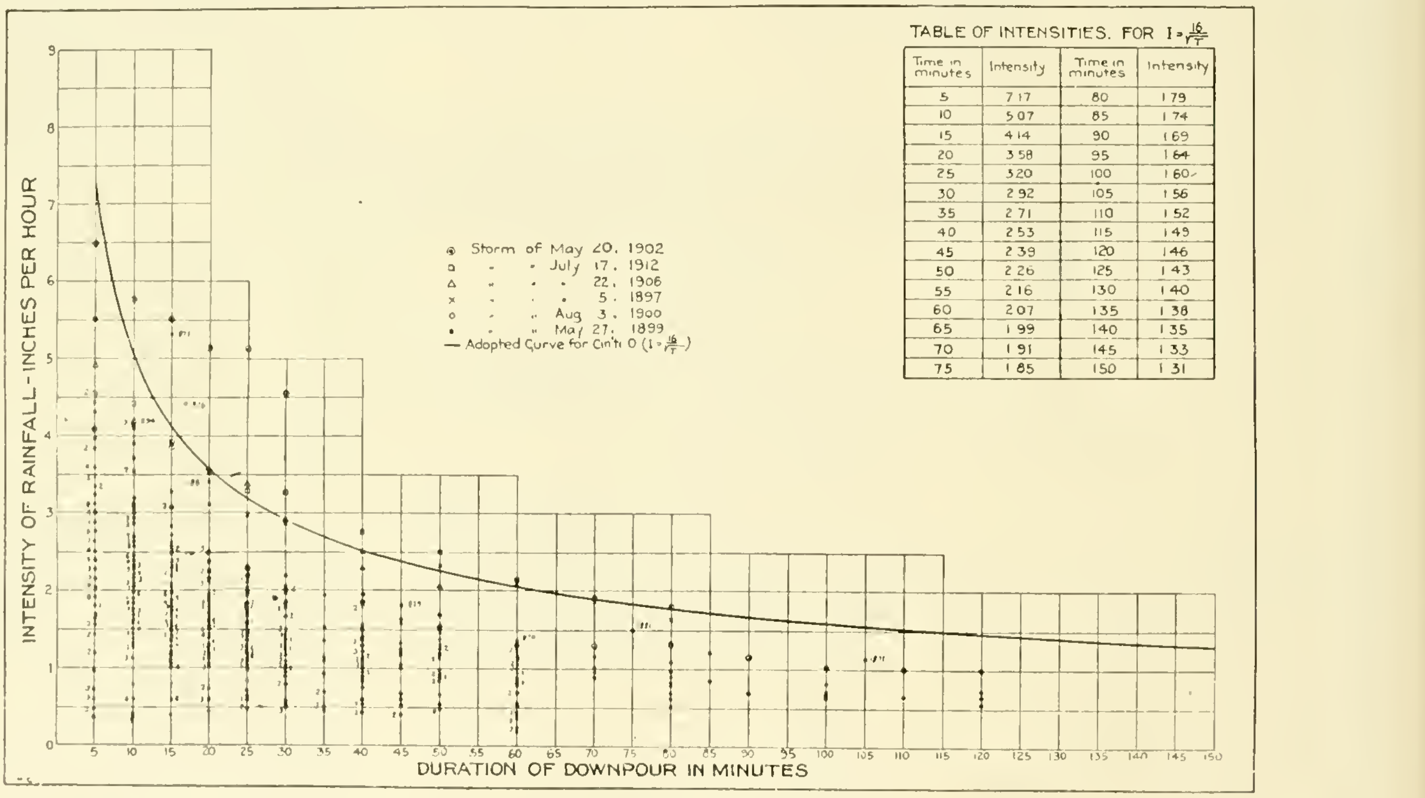


TABLE OF INTENSITIES. FOR $I = \frac{16}{\sqrt{T}}$

Time in minutes	Intensity	Time in minutes	Intensity
5	7 17	80	1 79
10	5 07	85	1 74
15	4 14	90	1 69
20	3 58	95	1 64
25	3 20	100	1 60
30	2 92	105	1 56
35	2 71	110	1 52
40	2 53	115	1 49
45	2 39	120	1 46
50	2 26	125	1 43
55	2 16	130	1 40
60	2 07	135	1 38
65	1 99	140	1 35
70	1 91	145	1 33
75	1 85	150	1 31

Fig. 1. Diagram Showing the Relation Between Intensity and Duration of Rainfall at Cincinnati from 1871 to 1912.

TABLE II.—RUN-OFF FROM IMPROVED AREAS IN TYPICAL COMMERCIAL DISTRICT.—(WALNUT STREET).

Area—30.4 acres.
Character of District—Commercial. Area covered by office buildings, stores and other business houses. No undeveloped area.
Soil—Sand and gravel.

Character of improvement.	Area in sq. ft.	Per cent of total area.	Estimated degree of imperviousness.	Equivalent imper. area sq. ft.	Per cent total area impervious.
Roofs	...	.....	..	....	59.8
Public and commercial	881,200	66.5	.90	793,080	....
Interior walks	....	.....	...	....	0.8
Brick	7,500	0.6	.40	3,000	....
Cement	10,000	0.7	.75	7,500	....
Street walks	....	.....	...	....	8.1
Brick	6,100	0.5	.40	2,440	....
Cement	139,300	10.5	.75	104,475	....
Street pavements	....	.....	...	....	16.4
Asphalt, brick, wood block	145,500	11.0	.85	123,675	....
Granite block	111,400	8.4	.75	83,550	....
Macadam and cobble	23,221	1.8	.40	9,290	....
Total	1,324,221	100.0	...	1,127,010	85.1

Impervious coefficient for district, 85.1%.

*These values are adapted from those suggested by Mr. Emil Kuichling in the Kuichling and Bryant "Report on Back Bay (Boston) Sewers."

sewers determined in the usual manner. A closer estimate can then be made of the time factor and, if found necessary, a second calculation made of the sizes and grades.

Intensity or Rate of Rainfall.—The use of the "rational" method of analysis requires a thorough study of the local rainfall records for the purpose of determining a rainfall intensity curve which shall define the limit of excessive storms for which provision should be made in the design of storm sewers. By means of this curve the rate of rainfall can be determined for any period of time.

Rainfall Records.—Rainfall records for Cincinnati have been kept by the U. S. Weather Bureau from 1871 to date. Through the courtesy of the local forecaster of the weather bureau, these records have been examined and data relating to significant storms taken from them and made available for use as the basis of the rainfall studies.

The U. S. Weather Bureau published in 1912 the total monthly rainfall for each year from 1871 to 1912 inclusive, together with the total annual precipitation and the mean precipitation for each month, and for each year. These data were available but were not of particular use in the rainfall studies made in connection with the sewerage problem.

Unpublished Data Furnished by U. S. Weather Bureau.—1. The records of heavy rainfalls having a total precipitation of 1.5 ins. or more, in 24 hours, or having a rate of 1 in. per hour for shorter periods, with extracts from the daily journal, from 1871 to 1897, were tabulated.

2. The hourly amounts of precipitation during the storm of March 12 and 13, 1907, during which 7.19 ins. of rain fell during 18 consecutive hours were tabulated. While this was an exceptionally heavy rainfall extending over a long period of time, it did not give the very high intensities which make it of great significance in the problem of sewer design.

3. The accumulated amounts of excessive precipitation from July 1, 1897, to July 31, 1912, for short periods of time from 5 minutes to 120 minutes were tabulated. From 1897 to 1904 the amounts were tabulated for each 5 minutes for storms in which the rate of precipitation equaled or exceeded 0.25 in. in any 5 minutes or 0.75 in. in 1 hour. After 1904 the amounts are tabulated for all storms during which the rates of precipitation equaled or exceeded in any period those given in Table I.

The data mentioned in the preceding paragraph were compiled from the charts taken from the automatic recording rain gage at Cincinnati. These are the storms which form the most reliable basis for studies of intensity of precipitation and, while the older records furnish some valuable data the chief reliance must be placed on the records of the last 15 years.

Rearrangement of Data Furnished by Weather Bureau.—The table mentioned under paragraph 3, above, gives the amount of accumulated rainfall by periods ranging from 5 to 120 minutes, from the beginning to the end of excessive precipitation but does not give the maximum rainfall for each length of period indicated. A table was prepared to give the maximum precipitation for periods of from 5 to 120 minutes duration. In this table the precipitation given for any specified period was the maximum amount falling during a period of such duration. For example, during the storm of Sept. 13, 1910, the maximum quantity of precipitation during any 10 minutes was 0.41 in., whereas the amount which had fallen at the end of the first 10 minutes of excessive precipitation was but 0.21 ins., and the former figure does not necessarily include the latter.

The intensity of precipitation was calculated for short periods of from 5 to 120 minutes for each of the significant storms, the result of said calculations being tabulated. These calculations are based upon the maximum quantity of precipitation during the period specified, as indicated by the data in the tabulation mentioned in paragraph 3. The intensity tabulation includes data relating to 93 storms occurring from July 5, 1897, to July 17, 1912. These data form the basis of the studies which have been made to determine the rate of rainfall for which provision should be made in designing sewers and upon which the rainfall curve hereinafter described has been based.

An examination of all records from May 31, 1871 to July 17, 1912, indicates that there have been 63 storms during which the intensity for some period of time was equal to or in excess of 1 in. per hour and which produced a total rainfall of at least 0.5 in. during the stated period. The data relating to these storms are compiled in a table, entitled "Intensity of Precipitation During Excessive Storms at Cincinnati," and are arranged in order of descending intensities.

In another table entitled "Excessive Rainfall at Cincinnati," there have been compiled the

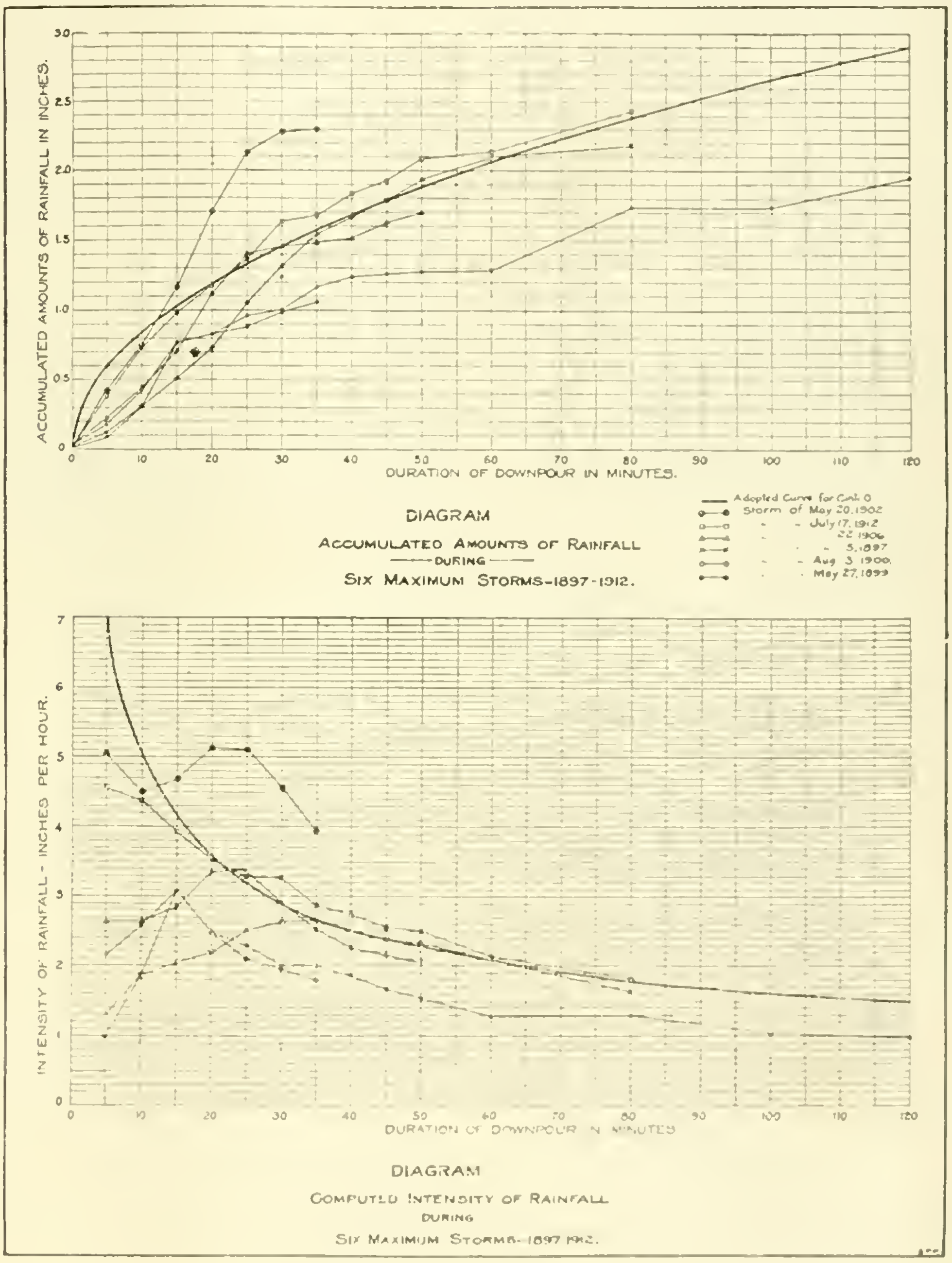


Fig. 2. Rainfall Diagrams for Cincinnati, Ohio, for Years 1897-1912.

TABLE IV.—RUN-OFF FROM IMPROVED AREAS IN TYPICAL COMBINED TENEMENT AND INDUSTRIAL DISTRICT (BADER STREET)

Area—35.6 acres.
Density—55 persons per acre.
Character of District—Tenement and Industrial.
Soil—Clay and sand to gravel.

Character of improvement	Area in sq. ft.	Per cent of total area	Estimated degree of imperviousness	Equivalent impervious area sq. ft.	Per cent total area impervious
Roofs					21.[illegible]
Residences	289,200	18.6	90	[illegible]	
Public and commercial	66,800	4.3	90	60,120	
Barns and sheds	79,200	5.1	75	[illegible]	
Interior walks					[illegible]
Brick	[illegible]	[illegible]	40	[illegible]	
Cement	[illegible]	1.[illegible]	[illegible]	16,9[illegible]	
Street walks					[illegible]
Brick	18,200	1.1	40	[illegible]	
Cement	78,100	5.0	[illegible]	[illegible]	
Street pavements					6.[illegible]
Macadam and cobble	[illegible]	[illegible]	[illegible]	[illegible]	
Unimproved					6.7
Unpaved yards and lawns	692,436	44.7	15	103,8[illegible]	
Total	1,550,736	100.0		688,150	44.4

Impervious coefficient for district, 44.4%.

data relating to all storms of excessive total rainfall or excessive intensity, arranged by months. This table indicates that there have been 92 storms since 1871 when the total rainfall amounted to or exceeded 2 ins., or, where the intensity amounted to or exceeded 1 in. per hour for a period in which the precipitation was 0.5 in. or more. There are given in this table the total precipitation for the whole storm, as well as the amount of precipitation, duration and intensity of precipitation during the period of excessive rate of rainfall.

Diagram of Intensity and Duration of Precipitation.—On Fig. 1 the rate of precipitation of each of the excessive storms for periods of from 5 to 150 minutes has been plotted as has also a curve which marks approximately the upper limit of all storms, except that of May 20, 1902, which was of extreme intensity. It is unfortunate that records showing the rates of precipitation are not available for a period of more than 15 years, but the fact that there have been during this time four storms in which the intensity was practically that shown by the curve, indicates that such storms may occur at intervals of from 3 to 5 years. It has been deemed wise to use in the design of the storm sewers, a curve, represented by $i=\frac{16}{T^{0.5}}$ to include these storms, in which i=intensity of precipitation and T=duration of period under consideration. It has not, however, been deemed wise to include the extreme storm of May 20, 1902, which, based on present records is not likely to occur oftener than once in 15 years and probably will recur at even greater intervals.

TABLE V.—RUN-OFF FROM IMPROVED AREAS IN TYPICAL RESIDENTIAL DISTRICT (WALNUT HILLS—DISTRICT "A").

Area—291.1 acres.

Density—20 persons per acre.

Character of District—Residential. Well developed. Detached dwellings of two or more stories and in good condition; 2.3 acres open and undeveloped.

Soil—Yellow and blue clay, overlying beds of shale and limestone.

Character of improvement.	Area in sq. ft.	Per cent of total area.	Estimated degree of imperviousness.	Equivalent imper. area sq. ft.	Per cent total area impervious.
Roofs	....	.....	...	....	11.4
Residences	1,665,800	13.1	.90	1,499,220	....
Public and commercial	214,000	1.7	.90	192,600	....
Barns and sheds	171,900	1.1	.75	128.925	....
Interior walks	....	.....	...	....	2.2
Brick	73,200	0.6	.40	29,280	....
Cement	334,400	2.6	.75	250,800	....
Street walks	....	.....	...	....	2.9
Brick	127,000	1.0	.40	50,800	....
Cement	430,600	3.4	.75	322,950	....
Street pavements	....	.....	...	....	7.0
Asphalt, brick, wood block	630,200	5.0	.85	535,670	....
Granite block	133,100	1.0	.75	99,825	....
Macadam and cobble	612,300	4.8	.40	244,920	....
Gravel and poor macadam	46,500	0.4	.20	9,300	....
Unimproved	....	.....	...	....	9.4
Unpaved yards and lawns	....	.....	...	....	....
Tributary to paved gutters	7,246,800	57.1	.15	1,087,020	....
Not tributary to paved gutters	1,000,955	7.9	.10	100,095	....
Total	12,686,755	100.0	...	4,551,405	35.9

Impervious coefficient for district, 35.9%.

This curve is higher than the curves which have been adopted in several other places, but appears to be fully warranted by the rainfall data accumulated in Cincinnati since 1871 and particularly in view of the rainfall data secured by means of the automatic recording rain gage of the U. S. Weather Bureau during the last 15 years.

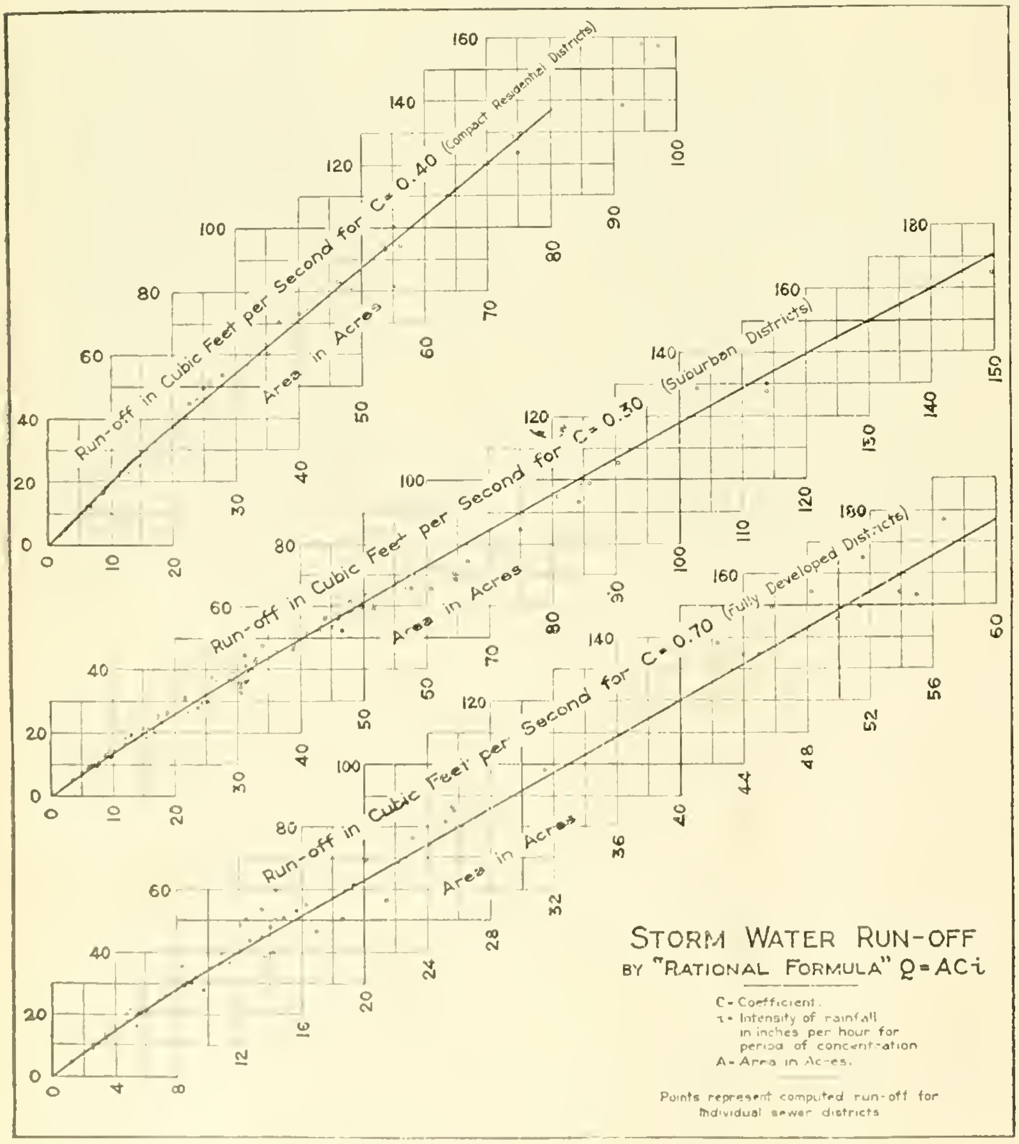


Fig. 3. Plat of Storm Water Run-off Curves for Cincinnati.

The topography of Cincinnati is such that the time of concentration in most of the sewers is very short, this making that part of the curve dealing with intensities for periods of from 5 to 30 minutes rather more important than that part dealing with intensities of periods greater than 30 minutes.

The intensities indicated by the curve have not been exceeded materially except by one storm, that of May 20, 1902. The storm of July 17, 1912, slightly exceeded the intensity indicated by the curve but the excess lies well within the limits of unavoidable errors.

In Fig. 2 the rainfall records for six maximum storms from 1897-1912 have been platted on two diagrams, the upper diagram showing the accumulated amounts and the lower diagram the actual intensity of precipitation for the successive periods of time. For purposes of comparison the curve adopted for use in designing sewers has been shown on these diagrams.

Ratio of Run-off to Rainfall.—Having determined upon the intensity or rate of rainfall for which provision should be made the next factor of importance is the determination of the proportion of the rainfall which immediately reaches the sewers in any section of the city. This proportion of the ratio of run-off to rainfall is relatively difficult of determination, not only for present conditions but also for future development. Some of the factors which influence its determination are, the extent of improvement and development within the area, the surface slopes, character of the soil, frequency of storm inlets as well as the capacity of the tributary or lateral sewers to carry off the storm water.

Although the conditions affecting this ratio vary between wide limits in the many different localities it was deemed sufficient to classify each section of the city as mercantile, tenement, industrial, residential, or suburban and then determine the run-off values for districts typical of each.

EFFECT OF IMPROVEMENTS ON RUN-OFF.

The factor of greatest importance is the extent to which each of these districts is improved and developed by buildings and pavements. To furnish the data bearing on this point surveys were made in the following districts which were selected as representing typical conditions:

1. Mercantile district. (Walnut Street Sewer.)
2. Tenement district. (Densely populated, Clark Street Sewer.)
3. Combined industrial and densely populated district. (Bader Street Sewer.)
4. Residential districts. (Walnut Hills, Bloody Run Sewer.)

In each of these districts the area covered by roofs, pavements, walks, yards, lawns, and the undeveloped territory were ascertained and to these different classes of surfaces run-off values* were applied and a final value determined upon for that district. The nature of the results obtained by these surveys and computations are shown in Tables II to V.

Sewer Gagings.—In addition to analyses of the surfaces of these selected typical districts, and the determination of the proportions of previous and impervious areas, gaging stations were established in the sewers draining these districts, for the purpose of furnishing data which will serve both as a check on the method of analysis used and the relation between the rate of rainfall and the storm flow in the sewer.

To gage the storm flows in the mercantile district an automatic clock-recording sewer gage was placed in a side chamber of a sewer in that district. This gage registers on a chart the depth of water flowing in the sewer during the progress of a storm. At the same time records of the rainfall are obtained by the Government rain gage located on the postoffice building, which lies within the area drained by this sewer. For the residential district a similar sewer gage was placed in a side chamber of a sewer in that district. In order to obtain the rainfall records within this drainage area, an automatic tipping bucket rain gage of the Frieze type was installed by the city.

The storm flow in the Clark Street Sewer, in the tenement district, is measured by means of a gage of the maximum or flood-flow type, consisting of a staff with small bottles attached on either side which are filled as the water rises in the sewer, the highest bottle filled registering the maximum depth of flow. Other gages of the same type were set at manholes in six other sewers.

These gages have been maintained since June 1, 1913, but the records obtained thus far have been too meager to warrant any deductions, especially since the storms of the last five months have been of very moderate intensity and of short duration. To obtain conclusive results from storm flow gagings it is necessary that they be extended over a long period of time in order that a sufficient number of records of storms of great intensities and sufficiently long duration may be secured. Such records will be of the greatest importance in the design of storm sewers for Cincinnati, and it is therefore to be hoped that these gagings may be continued for several years.

Run-Off Diagram.—Before the analyses of the improvements in typical districts were completed designs were in progress for new and relief sewers in the various parts of the city. In these designs the rational method was used and the following co-efficients were adopted for the determination of the amount of run-off to be provided for

C=.30 for suburban districts where little future development may be expected.

C=.40 for residential districts.

C=.70 for densely populated and completely developed tenement and small block districts.

The amounts of run-off which were determined in the course of the numerous designs have been plotted and curves projected by the use of which run-off quantities can be determined directly for any area in question without otherwise laborious computation work. These curves shown on Fig. 3 have been used for the later preliminary studies. For completely developed commercial districts higher co-efficients should be used.

METHOD OF STUDY.

With the completion of these preliminary studies, and decision as to method of design, the preparation of plans for new and relief sewers was at once started, the problems generally recognized as most urgent being attacked first.

Data Available.—The complaints received were used as an index to the localities or districts in most urgent need of relief sewerage. For the study of these districts it was necessary to have at hand the records of existing sewers, as well as the proper portions of the topographic map. In so far as possible the two branches of the Division of Sewerage Investigations which were carrying on these surveys, directed their efforts in such a way as to secure the necessary data as soon as possible. But the fact that such data were not at once available in the various parts of the city requiring sewerage plans has been a handicap to the progress made in these relief investigations.

Procedure.—To determine the cause of cellar floodings or other defective drainage and to recommend the most suitable relief measures, necessitated an exhaustive study not only of the sewers in the immediate vicinity of the flooding but in the adjacent territory as well, inasmuch as it was often necessary to divert portions of the drainage into adjacent sewer systems. For this purpose all existing sewers and inlets were plotted on the portion of the topographic map covering the district under investigation. The district was then subdivided to such an extent that the drainage to each inlet could be determined. The data were then tabulated in a manner which indicates at a glance at what points the existing sewer is of insufficient capacity, as well as the sizes and grades necessary for the proposed relief sewer. To supplement these studies a house to house canvass was then made to determine the extent of the territory affected by defective sewerage as well as to serve as a guide in the recommendation for the proposed relief.

The flooding of cellars by backwater from sewers, and other causes of defective sewerage have been traced to one or more of the following causes:

1. The improper use of sanitary systems of sewers by the connection of roof leaders and street inlets.
2. Inadequate provision for storm drainage in sewers intended to receive it.
3. Defective construction of combined sewers.

The problem of providing relief for such conditions was, therefore, somewhat different from the design of new sewerage, inasmuch as it is obvious that the existing sewers should be used to their fullest extent. In some cases sufficient relief could be obtained by replacing the local sewer with one of larger capacity, in others it was necessary to cut off and divert portions of the drainage into adjacent systems, while in some extreme cases it was found necessary either to reconstruct the sewer throughout its entire length to the outlet, or to provide an additional storm sewer. The latter alternative, however, was rarely possible, as it is generally necessary to place the proposed sewer at the same depth as the existing sewer in order to intercept the tributary or lateral sewers. Studies for relief sewers were carried to a point where it was possible to recommend the best and most economical methods of relief, as well as to prepare preliminary plans, estimates and reports.

General

Method of Making Sieve Analyses of Gravels.

In proportioning concrete it is quite important that the ratio of sand to large aggregate be accurately determined. Also in the examination of gravels for use as surfacing material on roads an accurate knowledge of the composition is of value.

The method of analysis in use in Illinois and Iowa is described in Bulletin No. 34 of the engineering experiment station of the Iowa State College by T. R. Agg and C. S. Nichols and is given here.

The first step in the investigation of a gravel is to determine the relative amount of sand and stone and the grading of each. This is done by means of the sieve analysis. For important work upon which a large amount of material is to be used it would be desirable to use ten sizes of sieves for the sand and the same number for the stone. But for work upon which only a few hundred cubic yards of material is to be used it is unnecessary to make such a complete analysis of the gravel. A small portable testing outfit will usually enable the user to make all of the examination required.

Gravel Testing Outfit.—In Fig. 1 is shown a gravel testing outfit which was developed by the Illinois Highway Commission, A. N. Johnson, highway engineer, and is used by the inspectors of the Commission. Similar outfits can be made by any tinsmith for about $3.50.

The outfit is made of galvanized sheet iron and galvanized screen wire and consists of the following parts: (b) a ⅛-in. sand screen, (c) a ¼-in. sand screen, (d) a ½-in. screen for stone, (e) a measuring vessel 4 ins. in diameter and 10 ins. deep. It has a cover which is shown lying beside the vessel. (f) A tube for protecting the glass graduate used for clay tests. The capacity of the tube (f) should be 50 per cent that of the measuring vessel (e), i. e., in the ratio of sand to stone commonly specified. (g) A glass graduate 12 ins. high and about 2 ins. in diameter which is used for the clay test.

The tube which protects the glass graduate fits inside the ⅛-in. screen and the three screens nest together and fit inside the measuring vessel. The outfit, packed for carrying, is shown at (a).

Sampling Gravel.—Obviously, the whole value of a test depends upon securing a representative sample for analysis. The sample should, therefore, be taken with great care. If taken from a stock pile of material, a quantity of gravel should be taken from a number of different places in the pile and the amount thus secured carefully mixed and spread into a circular pile and then reduced to the proper size for testing by repeatedly quartering. Several samples thus tested will indicate the accuracy of the results being obtained. If the sample to be tested is taken directly from a pit, the appearance of the pit must be carefully studied and the sample secured by taking portions from different places in the pit in such a manner as to obtain a representative sample, and one which will be similar to the material which will eventually be hauled from the pit. These preliminary samples will be sufficient to indicate the general character of the materials. It will be found necessary to make frequent tests upon the gravel as the work progresses if its quality varies to any appreciable extent.

Sieve Analysis.—Having collected a sample of the gravel which is thought to be representative, the large sheet-iron measure, e, in Fig. 1, which is 10 ins. high, should be filled with the gravel to be tested and shaken down until no more can be added. If the test is for the determination of the proportion of ¼-in. sand in the aggregate, then the material contained in the measure is screened through a ¼-in. screen and the two fractions thus obtained placed successively in the measure, thoroughly shaken down and the amount recorded. Since the container is of uniform diameter, the volume will be proportional to the height of the material in it. For example, if it is found that there are 6 ins. of sand in the container, the percentage of sand would be 60 and if there were 5 ins. of the stone the percentage of stone would be 50. These percentages, when added together, give 110 per cent as will be noted.

In making the sieve analysis by volumes it will always be found that since the fine aggregates fill in the voids between the coarse aggregates when the two are mixed together, the sum of the volumes of the two when separated will be somewhat greater than the volume of the two mixed. Therefore, it will be found that the percentage of the two fractions added together will range from 110 to 125, depending upon the character and relative percentages of voids of the two parts into which the pit run gravel has been separated.

If it is decided in addition to determining the proportion of sand in the bank gravel, to determine the grading of the sand, then the sand itself may be screened through the ⅛-in. screen and the proportion of material which is retained on, and passes through ⅛-in. screen measured, which gives an indication of the grading of the sand itself. Likewise, it may be desirable to determine the grading of the coarse aggregates by screening it through the ½-in. screen. If the percentage of ⅛-in. sand is desired it can be determined by using the ⅛-in. sand screen instead of the ¼-in. By referring to the curves in Figs. 2 and 3 the grading of the sand can be checked.

Clay Tests.—An approximate determination of the amount of clay or loam in an aggregate may be made as follows: Fill the glass graduate, (*g*) in Fig 1, about half full of the aggregate which is to be tested and then add water until the graduate is about three-fourths full, thus leaving a space at the top of the graduate unfilled. The graduate is then shaken for at least 5 mins. so as to give time for the water to wash off all clay or loam from the particles of the aggregate. After the graduate has been shaken it is suddenly turned right side up and set aside to allow the clay to settle. The clay or loam will settle on the top of the aggregate and may be measured after the water clears. The layer of clay thus deposited will vary in thickness depending upon the length of time it is allowed to settle as it tends to become more compact the longer it stands. If the measurement is made soon after the test is completed, that is, after it has stood for only a few minutes, a larger proportion of clay will show than if the measurement is made after the clay has stood for some time.

It has been found by experiment that if the clay has been allowed to stand for three hours, the percentage will then be approximately four times the quantity that would be obtained if the clay should be removed and evaporated to dryness. Therefore, it may be assumed for ordinary practice that the percentage of clay at the end of 3 hours is 4 times the dry percentage, and if the limit of clay by dry measure is 3 per cent, then there may be permitted by the method outlined above, 12 per cent of clay in an aggregate. This method is not exact, on account of the fact that water containing clay has settled into the voids of the aggregate in the lower portion of the graduate, but will serve unless the test thus made shows a percentage of clay near the allowable limit, in which case the exact test should be made.

If it is desired to get exact results, all the clay must be washed out of the gravel, the washings collected together and evaporated to dryness. The clay content can thus be accurately determined.

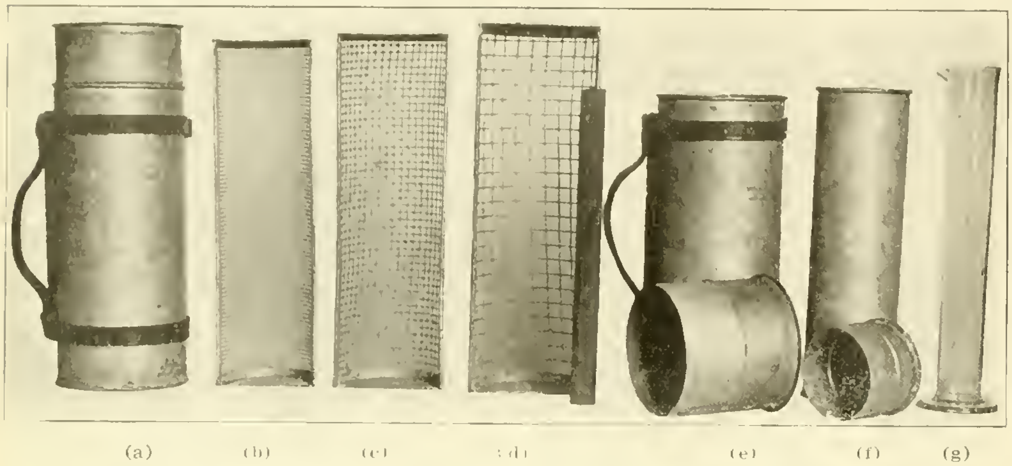


Fig. 1. Illinois Gravel Screening Outfit for Field Use.

(a) container, (b) ⅛-in. screen, (c) ¼-in. screen, (d) ½-in. screen, (e) measuring vessel, (f) protector for glass graduate, (g) glass graduate.

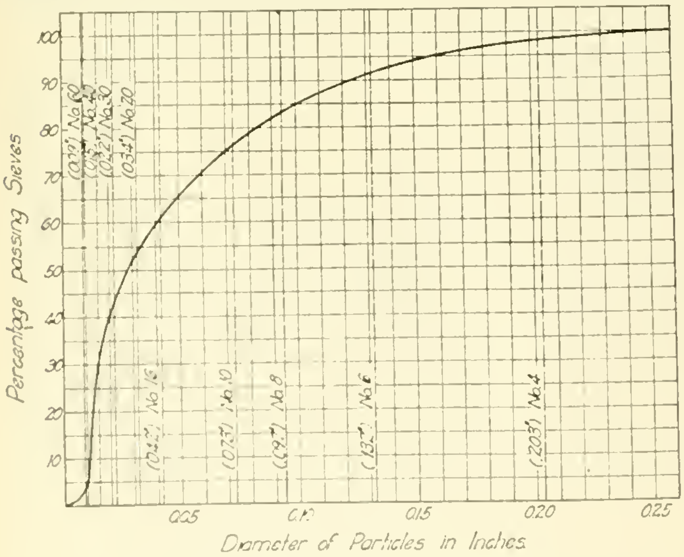


Fig. 2. Curve Showing the Variation in Size of Particles and Proportion of Different Sizes of Well-Graded ¼ In. Sand.

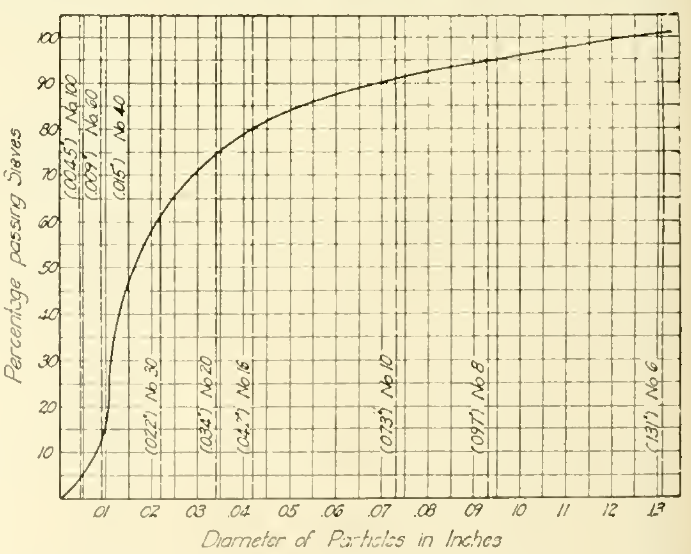


Fig. 3. Curve Showing the Variation in Size of Particles and Proportion of Different Sizes of Well-Graded ⅛ In. Sand.

Water Meter Statistics.—Three cities have reported on the number of meters in use on January 1 of this year as follows: New Haven, Conn., 3,000; Tampa, Fla., 1,621, an increase of 141 over January, 1913; Bloomington, Ill., 4,880, an increase of 490 over January, 1913

International Engineering Congress of the Panama-Pacific Exposition.

The International Engineering Congress, which is to convene at the Panama-Pacific International Exposition in San Francisco will hold its sessions from September 20 to 25, 1915, inclusive.

This is the third similar international congress that has assembled in this country, the two previous ones having been held at the Chicago and St. Louis Expositions. It is to be held under the auspices of the following five national engineering societies: American Society of Civil Engineers, American Institute of Mining Engineers, American Society of Mechanical Engineers, American Institute of Electrical Engineers, and Society of Naval Architects and Marine Engineers.

Colonel George W. Goethals, chairman and chief engineer of the Isthmian Canal Commission, will be honorary president of the Congress, and is expected to be present and preside at the first general meeting. Engineers of prominence throughout the world, whose names will be announced later, will serve as vice presidents. Both the general sessions and those of the various sections will be held in the great auditorium erected at the Civic Center in San Francisco.

Papers by eminent engineers will be read and discussed, and it is expected that attention will be confined exclusively to the progress of the science during the past decade and to its present status in all countries. The papers and proceedings will be published in volumes—probably ten in number, exclusive of a smaller volume containing the indexes, digests and general proceedings—of about 500 pages each. The volumes will contain papers as follows:

Volume 1, Panama Canal; 2, Waterways and Irrigation; 3, Municipal Engineering, Including Water Supply, Sanitary Engineering, Roads and Pavements; 4, Railways and Railway Engineering; 5, Materials of Engineering Construction; 6, Mechanical Engineering; 7, Phases of Contact Between Electrical Engineering and Various Other Branches of Engineering Work, presumably interlapping with Volume 6; 8, Mining Engineering and Metallurgy; 9, Naval Architecture and Marine Engineering; 10, Military Engineering and Miscellaneous Topics, including engineering education, aeronautics and other special features.

The completeness and method with which each one of these volumes will deal with its own particular topic probably can be best indicated by a concrete example. Take, as illustration, Volume 6, dealing with mechanical engineering. Its sub-topics will be divided into twenty-six sections, as follow:

(1) Recent progress and present status of foundry practice and casting metals; (2) Recent progress and present status of the art of forging; (3) Equipment, processes and methods for the boilershop; (4) Machine-shop equipment, methods and processes; (5) Automatics; (6)

Special processes for shaping and forming metals; (7) High temperature flames in metal working; (8) Industrial management; (9) Safety engineering; (10) Industrial Museums as an educational factor; (11) The steam engine of the year 1915; (12) The steam turbine of the year 1915; (13) The internal-combustion engine of the year 1915; (14) Motors of the Diesel type; (15) The Humphreys gas pump; (16) The steam boiler of the year 1915; (17) Refrigeration, (18) Pneumatics; (19) Lubrication and lubricants; (20) Water wheels of pressure type; (21) Water wheels of impulse type; (22) Hydraulic power developments and use; (23) Power-plant design; (24) Motor vehicles, passenger type; (25) Motor vehicles, utility type; (26) Motor tractors.

It has been so arranged that other congresses or allied interests shall immediately precede or follow it, thus dovetailing with its work, so to speak. For example, two weeks preceding the International Enginering Congress is the meeting of the International Technical Commission, which has established international units in electrical matters; immediately preceding the sessions of the engineers is the International Electrical Congress, under authorization of the International Electrotechnical Commission and under the direct auspices of the American Institute of Electrical Engineers; while the week following the engineers' Congress will be devoted to the International Gas Congress.

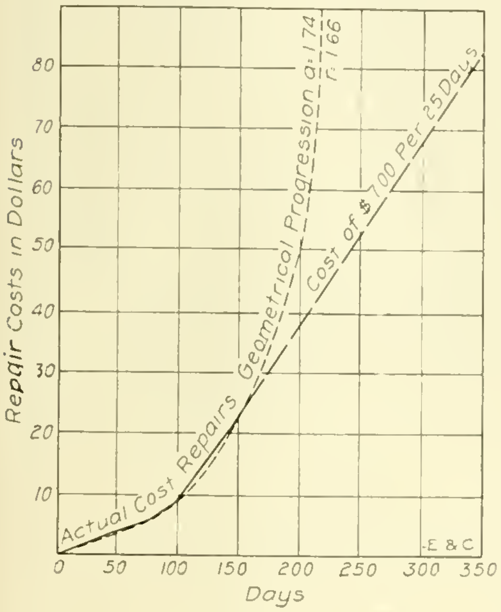


Fig. 1.

The various topics will be treated, in general or sectional sessions, by eminent engineers from all over the world, the number of representatives from each country being in some degree decided by the condition of the matter discussed in the various countries. In naval architecture, for instance, there will be ten treatises by Americans, ten by Englishmen, seven by Germans, three by Frenchmen, etc.

Of deepest interest to non-technical mankind, undoubtedly, will be the extensive consideration given to the Panama Canal, and the volume which will deal with that subject. It is expected that there will be one or more papers dealing with the economical aspects of the canal and its influence on trade routes; and, indeed, the subject will be very fully discussed.

In the volume of the Panama Canal to be issued the introductory chapter and the article on Dry Excavation for the Panama Canal will be written by Colonel Goethals, and each one of the twenty other articles will be written by a man prominent in the work, as follows:

Dredging in the Panama Canal—W. G. Comber, Terminal Works, Dry Docks and Wharves of the Panama Canal, H. H. Rousseau, Coaling Plants and Floating Cranes of the Panama Canal—F. H. Cooke; Meteorology and Hydrology of the Panama Canal—F. D. Willson, Design of Locks, Dams and Regulating Works of the Panama Canal—Col. H. F. Hodges, Method of Construction of the Locks, Dams and Regulating Works in the Pacific Division of the Panama Canal—S. B. Williamson, Design of the Lock Walls and Valves for the Panama Canal—L. D. Cornish, Design of the Spillways on the Panama Canal—E. C. Sherman; Gates of the Panama Canal Locks—Henry Goldmark, Electrical and Mechanical Installations of the Panama Canal—Edward Schildhauer, Emergency Dams Above Locks of Panama Canal—T. B. Monniche; Municipal Engineering and Domestic Water Supply in the Canal Zone—George M. Wells; Reconstruction of the Panama Railroad—Lieut. Frederick Mears; Aids to Navigation of the Panama Canal—W. F. Beyer; Geology of the Panama Canal—D. F. MacDonald; The Working Force of the Panama Canal—Capt. R. E. Wood; Purchase of Supplies for the Panama Canal—F. C. Boggs.

The widespread interest that is being taken in this congress is indicated by the fact that membership subscriptions already have been received from forty-six foreign countries, totaling about thirty per cent of the entire number, and they come from the most unexpected places. For instance, nine subscriptions have been received from the Dutch East Indies.

A membership subscription costs five dollars. It entitles its holder to all the privileges of the International Engineering Congress, the index volume and any other one volume that may be selected by the subscriber. It may be obtained by sending the five dollars to International Engineering Congress, 1915, Foxcroft Building, San Francisco, California, U. S. A.

Some Records of Rock Drill Repair Costs.

The following records of repair costs of rock drills are taken from a paper by C. K. Hitchcock, Jr., Manager, Lake Copper Co., Lake Mine, Mich., and published in the School of Mines Quarterly, Columbia University, for January, 1911:

TABLE I. REPAIR COSTS OF $2\frac{3}{4}$-IN. DRILLS

Drill.	Days in use.								
	25	50	75	100	125	150	175	200	225
A	$ 0.60	$ 1.25	$ 1.50	$ 2.40	$ 3.00	$ 7.05	$14.80	$11.40	$21.89
B	3.75	7.50	7.90	8.75	12.00	16.27	21.80	....	....
C	3.30	10.00	11.10	13.50	20.30	19.75	40.91	10.95	....
D	.90	1.25	4.55	5.30	28.39	28.39	28.39	....	....
E	.15	.40	1.50	1.50	5.80	6.40	9.20	9.85	13.20
F	3.80	7.40	8.00	8.08	8.08	12.1[illegible]	.....	.....	.....
G	.20	.50	1.10	1.75	8.00	14.95	16.20	20.08	27.20
H	.55	1.05	5.60	5.91	7.80	10.57	10.70	..	....
I	1.00	6.20	8.70	12.30	41.20	54.79	.....	.....	...
J	.70	.70	3.50	1.70	8.29	8.45	.....	.....	.....
K	.60	1.00	1.50	21.00	41.00	42.10	43.36	52.40	....
L	.55	1.00	2.20	9.10	12.00	12.38	21.30	....	.....
M	.85	1.20	4.40	7.70	8.76	10.27	18.14	22.70	....
N	3.65	4.10	4.85	5.05	8.50	50.50	52.01	.....	....
O	2.55	4.35	4.65	5.20	11.20	11.60	14.75	...	....
	$26.15	$47.90	$71.05	$118.24	$224.26	$329.30	....	....	.....
Average	$ 1.71	$ 3.19	$ 4.73	$ 7.88	$ 14.95	$ 21.95	...	...	...

The keeping of the costs to be discussed in this article was started with a group of 20 new drills. Of this number, five did not see enough service to make their record of value, therefore the data here given relates to 15 machines. They were of a well-known make, having $2\frac{3}{4}$-in. pistons, and were put to work during the spring and summer of 1912. The records were kept by the month, and the total repair cost of each drill, including both labor and material but not including mountings or connections, was plotted at the end of each month on a chart against the total number of days, of two shifts each, during which the drill had worked from the time it was put into commission. From this chart then was scaled the cost for each drill for each period of 25 days, as recorded in Table I. The average of these amounts is shown at the bottom of Table I, and is plotted in Fig. 1 by a solid line. The service varied a good deal, as will be seen, ranging from 150 days to 325 days each. This variation does not indicate poor service, but is explained by the fact that the drills were not all put to work at the same time, and also that the number of drills on hand was somewhat in excess of the actual number in use, so that some were always idle, whether in the repair shop or not. Good average results, therefore, were obtained for only 150 days, beyond which time the averages are incomplete and inconsistent, and lose all significance. It should be added that most of the smaller repair parts for the drills were made in the mine shops, but that the larger parts were bought from the makers of the drills. Also, when a machine was sent to surface, it was replaced at once by another, no serious attempt being made to keep the same machine in the same working place.

It is a curious fact, though probably only a coincidence, that the average repair cost for the first 150 days approximates closely the following geometrical progression, having 1.66 as a ratio: 1.71, 2.89, 4.80, 7.96, 13.22, 21.95. This progression is plotted as a dotted line on Fig. 1, the solid line representing the actual repair cost. Obviously, if the repairs should follow the dotted line to, say, 200 days or further, the expense of keeping up these drills would be so great as to indicate that they had practically gone all to pieces, which, of course, is contrary to general experience. I believe, therefore, that beyond 150 or 175 days the repair cost graph, if it could have been continued, would have become in time approximately a straight line, having such inclination as to represent a cost of $10 or over for each 25 days, provided all the machines were kept in commission. This approximation is based partly on the fact that the actual repairs between 100 and 150 days were about $7 for every 25 days, and partly upon the performance of another size and type of drill, which I shall now discuss.

The drills of the second group were larger, having a $3\frac{1}{8}$-in. piston. Eight new ones were put to work in 1911, and the records were kept in the same way as for the $2\frac{3}{4}$-in. machines. But after two of the drills had worked about 160 days, their cylinders cracked and they were put out of commission. The repair costs and service of the remaining six drills for two years is given in Table II, and is plotted in Fig. 2. The graph thus produced is more irregular than that for the smaller drills, but a straight line joining the two ends averages the irregularities fairly well and represents a cost of practically $5 per 25 days. Of course, this is hardly a fair comparison, and for the purposes of this discussion, it would have been desirable to have had the drills repaired and kept in commission.

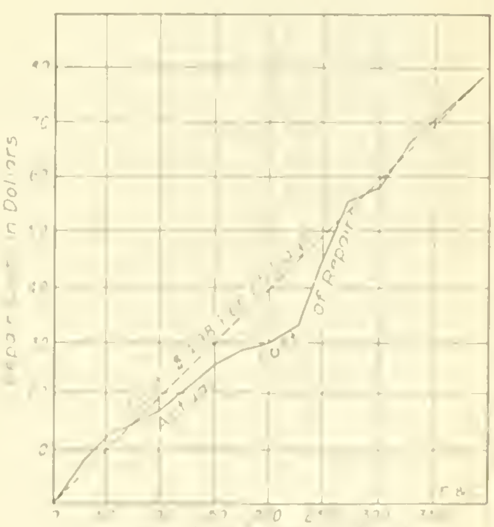


Fig. 2.

The immediate cost would have been between $60 and $80, with a resulting average increase of $8 or $10 at the 175-day period. How much of this increase would have shown as an increase throughout the remainder of the graph, is of course uncertain, but probably enough to have brought the average beyond 175 days up to $10 for every 25 days.

Again, from other data too incomplete to give here, obtained from such of another group of drills of the same size and type as had been in commission for from three to five years, the repairs amounted to $7.50 every 25 days. This rate, of course, is based on the best machines of the lot, the poor ones having been discarded. If all had been kept at work, there is evidence that the cost would have been nearly double.

TABLE II.—REPAIR COSTS OF 3⅛-IN. DRILLS.

	Days in use.							
Drill.	25	50	75	100	125	150	175	200
P	$ 2.00	$ 2.50	$ 4.00	$ 5.40	$ 8.00	$19.00	$22.00	$22.60
Q	3.00	11.30	19.10	22.30	27.00	34.00	40.20	41.30
R	6.10	14.20	18.20	19.60	24.50	28.60	28.90	28.90
S	4.20	5.20	6.20	9.20	18.10	18.20	18.30	21.50
T	28.00	33.30	36.10	37.00	38.10	41.50	44.70	45.90
U	.50	4.30	4.90	11.00	12.80	14.00	17.80	18.20
	$43.80	$70.80	$88.50	$104.50	$128.50	$155.30	$171.90	$178.40
Average	$ 7.30	$11.80	$14.75	$17.42	$ 21.42	$ 25.88	$ 28.65	$ 29.73
Drill.	225	250	275	300	325	350	375	400
P	$ 40.00	$ 64.20	$ 84.80	$ 88.10	$ 97.00	$ 99.50	$101.50	$102.00
Q	42.20	46.70	53.20	59.60	66.00	66.80	67.00	78.80
R	32.60	33.40	46.30	49.20	50.20	48.80	54.50	61.00
S	22.10	46.00	46.00	47.60	48.20	53.70	54.60	57.10
T	46.50	46.90	48.20	49.50	72.00	90.50	111.20	112.00
U	18.20	35.00	55.20	59.30	60.20	60.70	63.00	63.70
	$201.60	$272.20	$333.70	$353.30	$393.60	$420.00	$451.80	$474.60
Average	$ 33.60	$ 45.37	$ 55.62	$ 58.88	$ 65.60	$ 70.00	$ 75.30	$ 79.10
Drill.	425	450	475	500	525	550	575	600
P	$115.50	....	....	....	....	....	....	....
Q	79.50	....	....	....	....	....	....	....
R	75.00	$ 76.70	$ 77.30	....	....	....	....	....
S	60.20	62.70	62.70	$ 63.00	$ 64.00	$ 67.70	$ 78.10	$ 78.30
T	113.80	113.80	114.00	114.50	....	....	....	....
U	64.00	70.00	73.70	74.40	98.80	....	....	....
	$508.00	....	....	....	....	....	....	....
Average	$ 84.67	....	....	....	....	....	....	....

The practical application of such data as are given should be obvious. For instance, having obtained or assumed the average repairs for a certain lot of machines at work on a given job, and having plotted it on a chart, it would then be an easy matter, even for the repair man, to see how things were going, simply by plotting the record of each individual machine, month by month, alongside the average. Some machines, at times, will show up badly beside the average, for some good cause, while others will compare poorly from some inherent defect, and will be a constant source of expense. By a study of the chart, in connection with the care of the drills, economy and efficiency could both be served, either by culling out the worst offenders, or by giving them such a thorough rejuvenation as would tend to cut down their maintenance expense.

A Device for Bending Rails and Heavy Bars for Concrete Reinforcement, Panama Canal.

Rail and bar bending devices, which can be easily provided on any work having heavy bars to bend for concrete reinforcement, are being used at Balboa, Canal Zone, for bending reinforcement for pier and dock construction. Figure 1, redrawn from the "Canal Record," illustrates one of the machines. It consists of a base or table of trapezoidal shape, with a depth of 5 ft. and with its greatest base 6 ft. 4 ins. long, on which is mounted the mechanism for bending or breaking the rail. The base is of ¾-in. steel plate, set on 8-in. I-beams and braced beneath by plates of steel, ¾-in. thick by 5 ins. wide. Heavy studs, or rail stops, are set at each end of the base to hold the rail in place, while the pressure is applied.

The pressure is supplied by a 10-in. air cylinder, with a 10-in. stroke, mounted on one corner of the table. The piston of the cylinder is jointed to a heavy iron bar, 2 ft. 4 ins. long, the other end of which is set in a stationary pivot alongside the rail. Near this fulcrum is attached to the bar a wedge-shaped block of steel which fits between flanges of the rail. The forward stroke of the cylinder thrusts the wedge-shaped block against the web of the rail, midway between the rail stops on the sides of the table. It thus attains the maximum leverage for bending.

The strain thus set up is sufficient to bend the rail to a considerable curvature. In addition, when the rail is under the strain of pressure, it may be broken by a sharp blow from a sledgehammer, applied opposite the point of the wedge-shaped block.

The cylinder is supplied with compressed air, at the regular working pressure. Its operations are controlled by a ½-in. three-way valve. In order to afford relief of pressure and avoid blowing the cylinder head off if the operator should neglect the proper manipulation of the controlling valve, a row of 3/16-in. holes has been drilled in the cylinder at a point about two-thirds of the distance of the forward stroke.

The outfit weighs about 650 lbs., and is portable. It may be used in railroad construction, for the bending of rails, by connecting the cylinder to the air brake system of a locomotive or train. The cut made when a rail is broken, however, is not clean enough to allow pieces to be joined up properly, in laying track, without finishing them off with a cold chisel.

The system has been very satisfactory in operation at Balboa. A gang using it can bend rails much faster than by any other simple field process, and can break rails a great deal more rapidly than by using the familiar "cold cut" method, of striking repeated sledgehammer blows on a chisel or pointed hammer held against the side of the rail.

A substitute for the forge and anvil for the bending of steel rods to be used as reinforcement for concrete, also has been placed in use at Balboa, in connection with the construction of the quay wall and pier of the permanent terminals.

It is similar in principle and general form to the apparatus for bending rails. The rods are bent by the thrust of the piston rod of an engine cylinder, while they are held in place on a table in front of the cylinder. The table is a steel plate, bored with number of holes in which movable steel pegs, or studs, are set according to the point and degree of bending desired.

Two such equipments are in use. One uses a single cylinder, 10 ins. in diameter, which was recovered from an old French locomotive. This outfit places a single crimp in the ¾-in. rods which are used in the lengthwise beams of the construction, and the cylinder has sufficient power to bend three or four of the rods at the same time. The second equipment employs two 14-in. cylinders, taken from the dumping mechanism of a 2-cu. yd. concrete mixer, which are mounted on opposite sides of the table. These make, at one placing on the table, opposite bends in the larger rods which are used in the main girders. They bend, without difficulty, rods as large as 1¼ ins. in diameter. All the cylinders are operated by compressed air, under pressure of about 80 lbs. to the square inch.

By the use of these equipments, five men are able to bend in an hour as many rods as they could bend in an 8-hour day by the former process of heating them in forges and bending over anvils. As the quay wall and pier require a total of approximately 4,000,000 lbs. of bent rods in their reinforcement, the saving in time and money by the use of these equipments is considerable. The cold-bending process, instead of lessening the strength of the steel, is believed to conserve it.

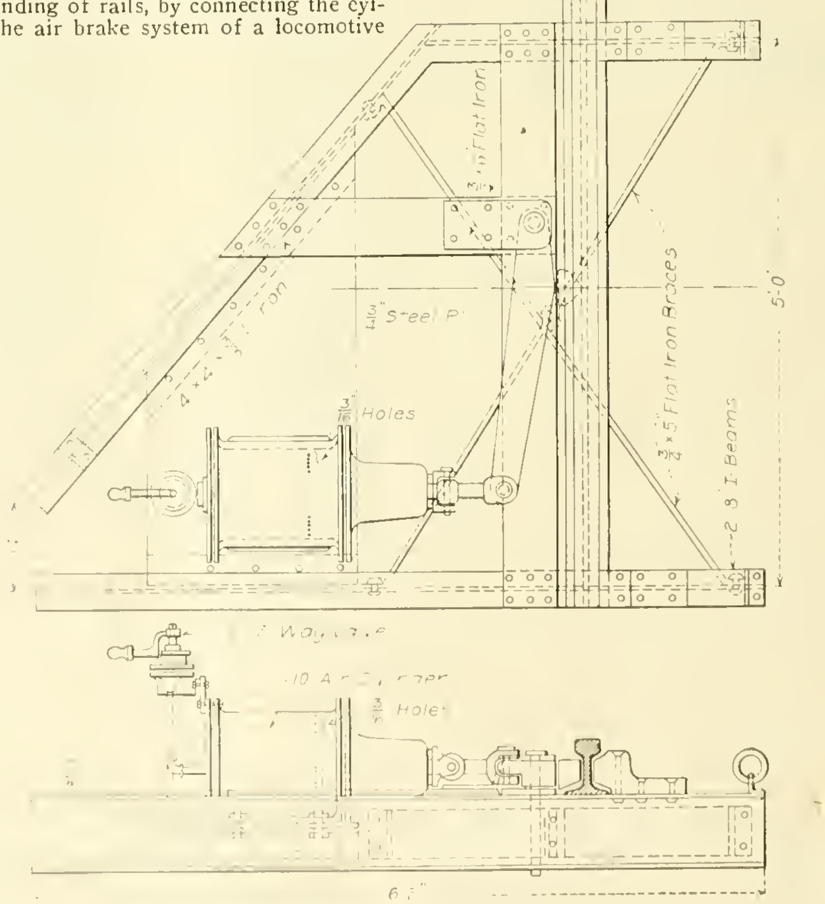


Fig. 1. Bending Tool for Reinforcing Bars.

Engineering and Contracting

Devoted to the Economics of Civil Engineering Design
and to Methods and Cost of Construction

Volume XVI. CHICAGO. ILL., JUNE 10, 1914. Number 23.

Preparing Calls for Bids to Encourage Bidders and Close Bidding.

What may be the duty of the engineer calling for bids to assist the bidder to all information that will aid him to set close prices is conceived differently by different engineers. One knows of some engineers who deny the contractor any aid outside the formal specifications and plans. Between this point of minimum help and the extreme of opening to the bidder all knowledge had by the engineer there are a multitude of degrees of co-operation. Practice offers nothing definite and opinion is its twin and both have some justification, but we give here an illustration of complete co-operation between the engineer asking for bids and the contractor who bids and we propose to approve the example.

The illustration selected is the call for bids for building the Bogue Phalia Mississippi drainage works. In the formal plans, specifications, bidding forms, contract · forms, and forms for various bonds, this call for bids does not vary from similar documents generally. Its exceptional feature is the information it offers the bidder in addition to these formal items. The following is an abbreviated summary of the information proffered: Complete copies of the engineers' report are available to the bidder. An engineer is provided to visit with the bidder the proposed work and explain its nature and answer inquiries. The railway connections are described and information is furnished of available fuel and water supplies. Estimates are given showing for each ditch dimensions, yardages, areas, amount of clearing and cost. Attention is directed to work calling for special plant, and suggestions are made of available methods. The character of this special information is indicated by the quotation which follows:

At the time of making the plans for the Bogue Phalia Drainage District it appeared that difficulty would perhaps be encountered in letting Contract No. 7 at reasonable rates, on account of the unusual size of the ditch. In order to indicate to contractors the possibilities for constructing this work the landowners within the district, at the suggestion of their engineers, made a contract with the manufacturer of a tower excavator to the effect that he was to install one of these machines on this ditch and to operate it for about a month. The landowners were to pay 10 cts. per yard for the earth excavated, and to purchase the machine for $3,500 in case it should maintain the required rate of excavation. The machine fulfilled the requirements satisfactorily, and was purchased by the landowners. It is now on the location of the ditch, and will be sold at its purchase price of $3,500 to any contractor taking this work.

The machine was operated for 216 hours between June 19 and Aug. 4, 1913, by one shift of men, during which time there were excavated 25,819 cu. yds., or at an average rate of 117 cu. yds. per hour. The total cost of operation is as follows:

	Total cost.	Unit cost.
Labor and board	$ 722.75	$0.028
Livery and horse hire	85.00	.....
Coal (85 tons at $3.45)	[illegible]	0.0111
Gasoline and oil	37.06	.....
Repairs	341.12	0.0132
Miscellaneous	1.[illegible]0	.....
	$1,180.48	......
Total pay yardage moved	25,819 cu. yds.	
Unit cost per yard	$0.0573	
Total time machine was in actual operation	215 hrs. 55 mins.	

Under item noted as "Repairs" about one-half was the cost of a new cable. Before undertaking this work the builder of the machine gave the officials of the district the right to construct other similar machines for the work upon charge of a nominal royalty. The ditch constructed is 80 ft. wide on the bottom, about 17 ft. deep, with 2 to 1 slopes and berms of 30 ft. As noted above, the unit cost per yard for this work was about 5.75 cts.

The greater significance of this quotation and of the summary preceding it does not lie in the specific information given. It exists in the spirit of co-operation which it indicates. The engineer by making available to the bidder all his knowledge of construction conditions accepts the bidder as a co-worker entitled to joint knowledge with the engineer of all facts that may bear on the cost of performing the work. We believe that this is a relationship which the engineer should always seek.

A call for bids has two main objects. One is to secure a structure of specified design and structural integrity. The other is to secure this structure at lowest cost. To attain these objects bidders must be induced and close bidding made possible. Eliminating construction uncertainties is the most effective way of attaining both bidders and close bidding. To illustrate the point: In the Bogue Phalia work the engineers felt in the instance of contract seven that uncertainty of how to handle the work might deter bidders and cause excessive prices to be named. By installing a special plant and operating it on a section of ditch, demonstration was afforded of the kind of plant that would do the work and of the cost of operating this plant. Uncertainty of possible methods and costs was eliminated in a large measure and bids could be made upon somewhat certain knowledge.

Sand Cement Grout.

Sand cement has been given much notice during the last year or two because of its adoption for important isolated structures of the U. S. Reclamation Service. The material is, however, new only in the sense that it has in the works mentioned been newly employed in great quantities. Twenty years or more ago the manufacture of sand cement was begun in the east with great expectations of its commercial development. Commercial success did not come but nevertheless there were a number of structures of no mean magnitude in which the combined material was used. One of them is mentioned in another column of this issue—the lock and dam between Minneapolis and St. Paul for the improvement of the Upper Mississippi River.

These works were begun sometime prior to 1895 and Portland cement and sand in equal quantities were ground together on the site to make sand cement which was used in considerable portions of the structures. It seems strange to read that the reason for using the combined cement on these upper Mississippi dams was the same essentially as that urged by the Reclamation Service for its use on the Arrowrock Dam—the high cost of straight cement due to freight charges. However, it is to be remembered that 15 years ago St. Paul was about as far by rail from big Portland cement mills as is the Arrowrock dam site today. Incidentally the behavior of the sand cement work at this Mississippi lock and dam furnishes interesting evidence of the durability of sand cement. It is stated that concrete of sand cement here, remains as solid and sound as similar concrete of straight Portland cement.

The comparison of combined cement and Portland cement is not, however, the subject that we set out to mention. Indeed no just comparison can be made based on such meagre data as are presented by the structures being considered. The feature that it is desired to emphasize is the particular merit that is claimed for grout of sand cement. This merit is that there is no segregation of the active cement and the inert sand as is the case due to difference in granular magnitude of ordinary sand particles and of cement particles where cement sand grout is injected. In sand cement mixture the sand particles and the cement particles are substantially the same size and being nearly of the same specific gravity there is small tendency toward segregation when water is added to make grout. There is another possible merit in sand cement grout. Where a fine, free flowing grout is wanted to penetrate fissured and porous rock, sand cement provides such a mixture at lower cost than will neat Portland cement.

The Systematized Maintenance of Country Roads.

There is a steadily growing conviction among students of highway problems that road officials have been somewhat hasty in adopting new methods and types of roads, and abandoning, without regard to the good points, the old system of statute labor. We do not believe in the statute labor system of road maintenance, but we do believe a system which has endured for several hundred years may be studied with profit.

The feature of the statute labor system which appeals to us is its extensiveness of scope and its inclusiveness. It covered every county, every township or borough, and every mile of road therein. It was fundamentally a maintenance system. How well the maintenance was accomplished is another matter. It was a system by which necessary work could be done immediately by the man on the job, the abutting resident.

The dragging system instituted in the state of Iowa about a year ago is a development of this idea and the results obtained under it are worthy of continued study. In effect it is a contract patrol system covering every mile of graded road in the state. It is admirable in this respect. While it is desirable to have surfaced roads and well constructed main arteries of traffic it is folly to neglect to maintain connecting roads.

The points mentioned herein have been recognized by the officials of the U. S. Office of Public Roads for several years, and the work of the division of economics of that office has been a study of these conditions. An interesting comparative study, which should lead to valuable results, has been recently undertaken by this division to determine accurately in various counties the methods and cost of building and maintaining all types of roads with different kinds of labor, together with methods of financing and accounting.

There is no question as to the need of systematized maintenance on all country roads. Many experiments of this kind are being tried at the present time and a number of them have proven expensive—New York state furnishes an example. It is to be hoped that engineers will devote the detailed study they should to this question, which study will aid materially in relieving road construction practice of many of the panaceas and cure-alls to which it is subjected at the present time.

The Lowest Bidder and the Lowest Responsible Bidder.

An editorial published in the current news section of this journal of Feb. 18, 1914, called attention to the fact that the lowest bidder and the lowest responsible bidder on a piece of work are often different contractors. An illustration was given in which the mayor of a small town insisted that a certain contract be awarded to the lowest bidder, although the engineer in charge suggested that the bid be rejected because it was obviously and very much too low. The mayor carried his point, the low bidder was awarded the contract and, owing to the lack of the requisite experience and working capital, soon abandoned his contract. The bondsmen, relatives of the contractor, refused to take up the work and suit had to be brought to compel them to do so. This, of course, meant a long delay in completing the work and heavy court costs. After the contractor abandoned his contract it was learned that he was an undertaker who, during a dull season in his business, decided to take up contracting as a side line. The editorial mentioned concluded that: "If some municipal officials would give less thought to the possible saving of a few dollars in the contract price and would devote more consideration to ascertaining if the bidder had sufficient capital, experience, ability and plant to carry out the work, there would be fewer 'undertakers' taking up contracting as a side line."

The foregoing rather extreme case serves well to illustrate one of the greatest faults in the application of the contract system of doing public work, namely, the awarding of contracts to lowest bidders who are not properly qualified to carry their contracts to successful completion. This evil is becoming better recognized and there is a growing tendency to award contracts to the lowest responsible bidder rather than to the lowest bidder. Theoretically this has always been the case, but actually the vast majority of contracts for the building of public works have been awarded to the bidders submitting the lowest bids regardless of their other qualifications. This practice has been by no means limited to small towns or even to municipal works; in fact the practice has been all too general.

The two quotations which follow indicate the growing disapproval among engineers of the practice here condemned. In discussing a paper on the state supervision of dams and reservoirs, presented before the New England Water Works Association, Mr. M. O. Leighton, chief hydrographer of the United States Geological Survey, in part said:

> Consider the contractor. It is undoubtedly true that many dam failures have been caused by poor work on his part. You know the predicament of a contractor who is losing money on his job, and how easy it is to consider the fact that he is losing money and therefore to allow a little indulgence here and there. To secure a good piece of work under such conditions, eternal vigilance is usually necessary, and few of us measure up to eternal vigilance. One of the best ways to obviate such a difficulty is to abolish that absurd and antiquated regulation which prevails, especially in public work, that the contract shall be awarded to the lowest bidder regardless of the cost and the difficulties and regardless of the real responsibility of the man. In 90 per cent of the cases, and probably more, the expense, trouble and delay caused by a defaulting contractor who is awarded a job at a too low price, more than exceeds the difference between the successful bid and the rational one.

Plans and specifications insure good construction, as a rule, only under rigid inspection Mr. Leighton's comments bring out a point which should be more generally appreciated by those who award contracts, that is, the human element must be considered in its relation to the rigidity of inspection when a contractor is losing considerable sums of money. The responsible contractor is one who will do good work even though at considerable financial loss to himself, but this type of contractor seldom bids too low in these days of basing bids upon carefully prepared contractor's cost estimates.

In his latest annual report, Mr. Charles A. Finley, superintendent of the Pittsburgh bureau of water, comments as follows upon the folly of awarding contracts to the lowest bidder, regardless of other considerations. To quote:

> The unfortunate and unwarranted delay in some of our new work calls attention to the necessity of more rigid determination of the ability of bidders to execute the terms of contracts. These delays have arisen from lack of management. The provision in our specifications, that the bidder shall have the necessary organization, capital, experience, etc., to execute the terms of the contract, should be made to mean just what it says. The fact that a bidder is the lowest bidder should not secure for him any consideration unless he had demonstrated the ability to execute such a contract.
>
> A competent contracting organization is just as essential to satisfactory results as is the engineering organization which designs and supervises the work.

Mr. Finley specifically names three of the most noticeable and annoying examples of delay in the completion of contracts in the recent experience of the Pittsburgh bureau of water. He states that in each of these cases bids were received from contracting organizations which had demonstrated their ability to comply with the progress required in the contract. They were not the low bidders, but in each case the city has lost money by awarding to the low bidders.

The popular adherence to the low bid regulation undoubtedly comes, in part, from the belief that a higher degree of honesty is present when contracts are awarded to low bidders rather than to relatively higher bidders. There are evidences that this simple faith is passing. Enough stories exploiting the smart tricks of those who graft on public works have been published in the popular press during the past few years to impress upon the more intelligent of the general public that if a disposition to graft is present the low bid regulation is easily surmounted. The public should now realize that the awarding of a contract to a high bidder for the purpose of "getting in" on the difference between his bid and the low bid is regarded as case A-sub-one among the simple forms of double-dealing under the contract system, and as much too crude a method to be employed with safety. So well is this ancient form of grafting understood that awarding a contract to anyone but the lowest bidder should be inferential evidence of the honesty of the award rather than the opposite.

The engineer in charge is best qualified to judge of the sufficiency of a bid and of a contractor's qualifications. If he sees reasons why the low bid should be rejected he should not hesitate to recommend that it be rejected.

The Quantity System of Estimating.

That our present methods of competitive estimating of building contracts are unsatisfactory is evidenced by the periodic movements which are started to change existing methods. The present attempt to establish the system commonly referred to as "The Quantity System of Estimating," or "Quantity Surveying," is attracting considerable attention and deserves careful consideration. Like most attempts at betterments the greatest difficulty at the start lies in the multiplicity of suggestions offered by those interested and in a misunderstanding of the purposes of the new system. Some extravagant claims have been made for the new movement, which, if taken seriously, will endanger its ultimate success. The system above referred to, which is advocated by the American Institute of Quantity Surveyors, possesses several advantages over the methods in present use, and its adoption should lessen the risks now taken by contractors.

Briefly stated, the main purposes of the new system, which is quite fully discussed in an article by Leslie H. Allen, published in this issue, is to furnish free of charge to each contractor who desires to submit a bid a "Bill of Quantities," together with the plans and specifications for the work. Such a bill is an accurate schedule of the quantities of labor and materials, together with a description of the work, required in the erection of a building. This bill is to be prepared at the owner's expense by an independent party, the "Quantity Surveyor," who is to be appointed by the owner, the architect, or the engineer. The adoption of this system should result in the following direct benefits: (a) It should enable large contracting firms to reduce their estimating staffs to a large extent and should permit the members of small firms to give a greater part of their time to actual construction work, thus reducing the overhead expense; (b) it should tend to place all contractors on the same basis in bidding, which should result in closer bids; and (c) it should not be necessary for contractors to add to their bids for "contingencies."

As has been pointed out by Mr. Allen in his discussion of the quantity system one of the greatest difficulties which confronts those who attempt to put it into operation is the lack of standards of measurement. Unfortunately, it is true that in no trade is there any definite agreement among contractors as to how the various items shall be measured. Before the new system can come into common use it will be necessary to adopt definite standards of measurement, and when this is done a long step will have been taken toward a solution of the problem. The author's discussion of the different procedures which may be followed in adopting the system deserves careful study. We agree with him that, due to the fact that the contractor is most vitally affected, it will be necessary for him to take the initiative in placing the quantity system in operation and to convince the owner that the latter will also be benefited by its adoption. Most owners appreciate the fact that the contractor is entitled to a fair compensation for efficient work, and we believe the owners' cooperation will be secured if they are convinced of the injustice of present methods and of the fairness of the system advocated. In principle, the new system seems fair to all parties concerned, and therefore its adoption should tend to create better conditions in the contracting business. As conditions vary greatly throughout the country it is evident that no set rules can be formulated for its adoption which will prove universally acceptable. The success of the movement in each city will depend to a large extent upon the common-sense methods devised by those in charge.

Official Opening of 30,000 HP. Plant near Columbia, S. C.—The Parr Shoals Power Co.'s new 30,000 HP. hydro-electric development near Columbia, S. C., recently completed by The J. G. White Engineering Corporation of New York, was officially opened on Decoration Day. Mr. E. W. Robertson, president of the Power Company, was the host. The completion of this development marks an epoch in the industrial development of Columbia, where natural facilities and fine location waited only for the introduction of cheap power for a boom in the progression of the city's enterprises. The Parr Shoals Co. is a subsidiary of the Columbia Railway, Gas & Electric Co., of Columbia, S. C., and the new plant supplements the steam and hydro-electric stations of the parent company, which are rated at 14,500 HP. The new station comprises a concrete dam 2,200 ft. long across the Broad River about 30 miles above Columbia. A modern fireproof power house has been provided for the power generating units. Provision has been made for the eventual development of about 30,000 HP. The present installation comprises five generating units, operating at 35 foot head, which will generate 18,000 HP. Current at 60,000 volts will be transmitted to Columbia over a double circuit, steel tower transmission line.

Buildings

Methods and Costs of Wrecking a 218-Ft. Reinforced Concrete Stack at Akron, Ohio.

(Staff Article.)

In connection with the extensive improvements now being carried on by the Quaker Oats Co. at its Akron, Ohio, plant it was found advisable to replace a reinforced concrete stack 218 ft. high and 7 ft. in inside diameter at the top with a new one of similar construction 250 ft. high and 11 ft. in inside diameter at the top. Space limitations (see Fig. 1) adjoining the enlarged power house made it necessary to erect the new stack on the site of the old one, and also made it inadvisable to follow the usual practice of letting the stack fall at one operation. The old stack was built about four years ago, the lower 11 ft. of the octagonal base being built by the Leonard Construction Co., in connection with the 1909-10 improvement at this plant, and the tapering circular stack by the Alphons Custodis Chimney Co., of New York, which also has the contract for building the new stack. Concrete stacks are so recent a development in the building field, and so few of them have been wrecked that the following data should prove of value to engineers and contractors.

OLD CONSTRUCTION.

The old stack was 12 ft. 9 ins. in inside diameter at the base, tapering to 7 ft. 0 in. at the top. The thickness of the reinforced concrete wall varied from 12 ins. at the base to 5 ins. at the top. Up to a height of 75 ft. a lining of radial tile blocks was provided, which varied in thickness from 8 ins. to 4 ins. The air space between the lining and the outer shell varied from 8 ins. to 2 ins. The upper 143 ft. of the stack were not lined. The only cracks found in the entire structure were confined to the 8-ft. bell at the top. Here there were six vertical cracks, the largest showing an opening of about ¼ in. and disappearing entirely within the depth of the bell. The stack was reinforced vertically with ⅝-in. square twisted rods lapped 3 ft. 6 ins. at all splices, the spacing varying from 3 ins. at the base to over 3 ft. at the top. These rods were wrapped from top to bottom of the stack with a single layer of American Steel & Wire Co.'s No. 23 wire mesh.

EQUIPMENT AND METHODS USED.

A temporary steel stack was first built (see Fig. 1), which enabled the power house to operate using natural gas as fuel, the soot being removed at the base of the tower.

During the wrecking of the stack the workmen were constantly engaged at its base in excavating for the foundations of the new addition to the power house, so that unusual precautions were taken to prevent accidents. A wooden tower was first built inside of the old stack. This tower was 222 ft. high, and was constructed of material taken from a dismantled concrete hoisting tower. Figure 2 (a) (p. 658) shows an elevation of a 18 ft. section of this tower, and Fig. 2 (b) shows a plan. These drawings give the general dimensions and the sizes of the various members. It will be noted that a ladder was built inside of the tower, as this location gave a greater feeling of security to the workmen. Resting platforms were placed at 18-ft. intervals. Those in charge of the work considered the installation of a hoist, to save the time and energy of the workmen, but this scheme was discarded. It was found that it took the workmen about ten minutes to ascend the entire height of the tower and practically the same time to descend. Each side of the tower, for its entire height, was sheeted with four 1 x 6-in. planks. Outriggers were fastened to the top of the tower, to which were attached regular bricklayers' hangers, which in turn supported the working scaffold. Figure (a) (p. 659) shows a plan of the outriggers and of the scaffold in its highest position, while Fig. 3 (b) shows the platform in its lowest position. These drawings indicate clearly the construction features of the scaffold and outriggers. The scaffold was enclosed at all times with a tarpaulin to prevent injury to the workmen below. The only accident on the job was occasioned by the falling of one of the long, vertical rods across the stack, which inflicted a slight scalp wound on a man working on the scaffold directly opposite. Figure 1 shows a view taken during the wrecking of the stack, and indicates the equipment used.

One man was kept busy with the work of lowering the scaffold as the wrecking progressed. The concrete was broken up with sledges, and was permitted to fall in the space between the inside of the stack and the tower, the sides of the tower being provided with the 1 x 6-in. guard strips shown in Fig. 2. One man gave his entire time to cutting the wire mesh with a bolt cutter and to tying the ends of the vertical reinforcing rods (to prevent their falling when loosened). Figure 4 (p. 659) shows the arrangement of the vertical rods and the wire mesh after the concrete was removed. The reinforcing rod, which is shown bent, was distorted to permit the workmen to gain access to the scaffold. The vertical rods were each 26 ft long. They were lowered to the ground outside of the stack, and were again used on other work. The rods were found to be bright and clean and were entirely free from rust. The stack was built in sections about 8 ft. high, and it was noticed in wrecking that all of the joints between sections presented a horizontal plane of clearage across which the concrete separated with a clean sharp break.

Every 48 ft. the outriggers were lowered and the upper portion of the tower then torn down. This operation required about one day. During this time the cutting was stopped for a part of the day on the upper half of the stack, while on the lower half the number of men was reduced but the cutting was not stopped. The broken concrete and other debris were wheeled in barrows to a fill for a switch track which was about 100 ft. distant, being handled during the nights and Sundays by two laborers. Except for a small amount of breakage the radial tiles were saved and will be used for the lining of the new stack. During the demolition of the lower 65 ft. of the stack it was necessary to use bull points, in addition to the sledges, in breaking up the concrete.

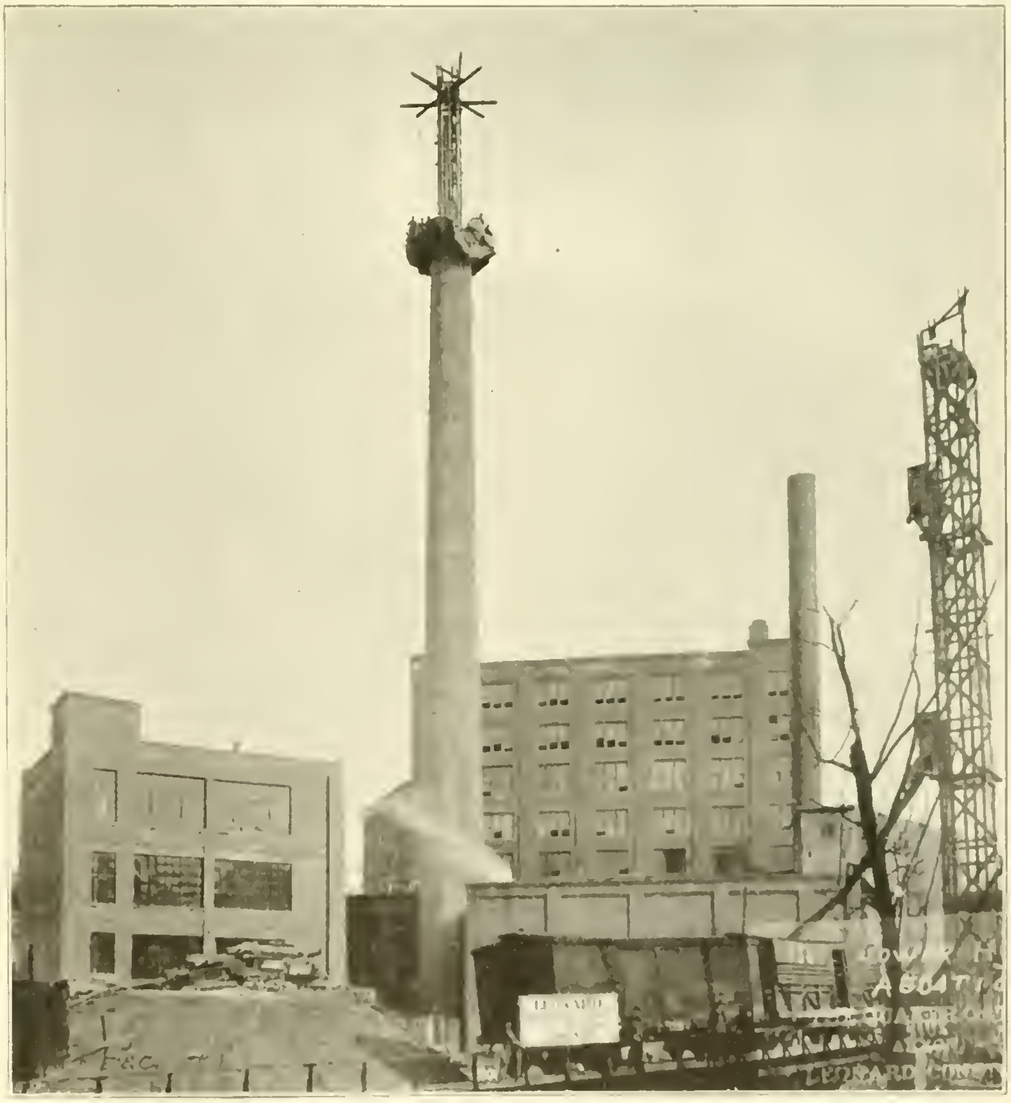


Fig. 1. View of Site Showing Reinforced Concrete Stack, Adjacent Buildings and Temporary Steel Stack of Plant at Akron, Ohio.

COST DATA.

Laborers were paid 30 cts. per hour and carpenters 40 cts. per hour. The total cost of wrecking 207 ft. of stack and 75 ft. of lining, exclusive of the liability insurance, was distributed as follows:

Item.	Cost.
Cleaning soot from below breeching....$	21.30
Lumber for tower and scaffold, 8,700 ft. B. M.	130.00
Erecting 222-ft. tower inside of stack, and building ladders, guard sheeting.	

platforms, outrigging and scaffold complete	211.27
Rental on bricklayer's scaffold hangers.	100.00
Cutting concrete and wire netting, the reinforcing rods and radial tiles being safed	649.45
Removing debris, 100-ft. haul, and stacking tiles and rods	68.25
Lowering outrigging and tearing down tower	202.26
Total cost, exclusive of liability insurance	$1,382.53

The liability insurance, covering employees and public, was $6.48 per $100.

Omitting the cost of clearing out the soot, and crediting the salvage, the total cost of wrecking the stack amounts to $5.20 per foot of height. Per cubic foot of concrete and tile removed the cost was 17 cts., or $4.50 per cubic yard. The salvage consisted of 8 tons of reinforcing steel valued at $220, 3,000 ft. B. M. of lumber valued at $30, and 1,200 cu. ft. of radial tile blocks valued at $60, a total of $310.

The tower was started on April 6 and was completed on April 18, 1914, with three days out for Sundays and rainy weather. The cutting and clearing away was entirely completed on May 15, 1914, after a further loss of five days for Sundays and rainy weather.

PERSONNEL.

The wrecking of the tower was accomplished by the Leonard Construction Co., of Chicago, in connection with the extensive improvements now being carried on by this company for the Quaker Oats Co. We are indebted to the former for the data contained in this article.

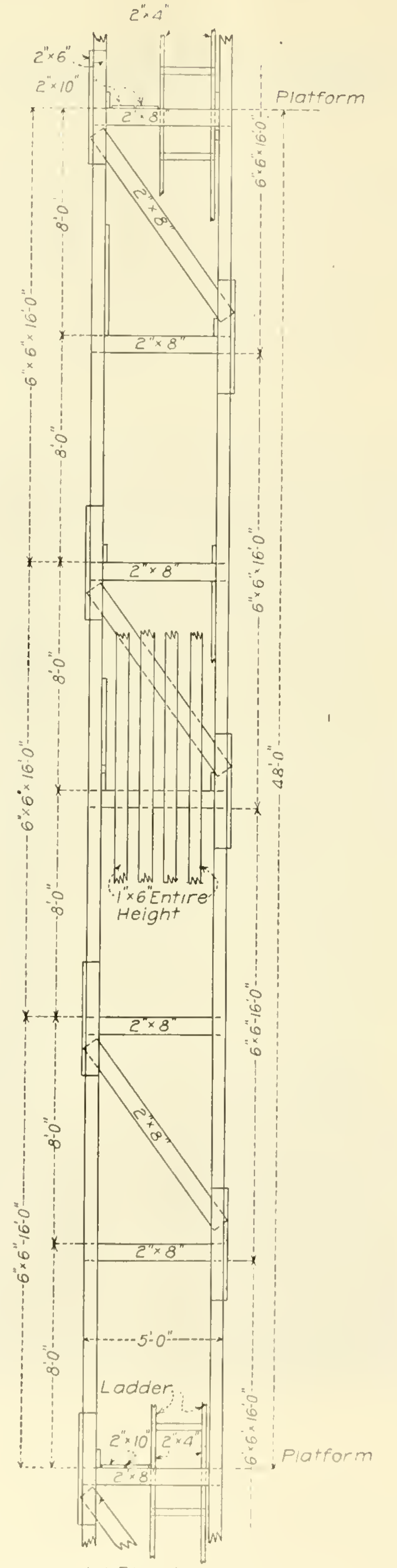


Fig. 2. Elevation and Plan of Tower Used for Wrecking Concrete Stack.

A Discussion of the Quantity System of Estimating.

There is little doubt that our present methods of competitive estimating of building contracts need revision. A complete revision of methods, however, will prove a difficult task, but this should not deter those interested from making a start. The "Quantity System of Estimating" is now receiving considerable attention from architects, engineers and builders. As much has been claimed for such a system, and as a clear understanding of what it means is necessary, we are giving below the essence of an address presented before the joint meeting of the Michigan chapter of the American Institute of Architects, the Detroit Engineering Society, and the Builders' and Traders' Exchange of Detroit, by Leslie H. Allen, of Boston. Mr. Allen is conversant with the system used in England, and has also had considerable estimating experienc in this country, being now connected with the Aberthaw Construction Co., of Boston. It will require much careful study before a satisfactory system is evolved, and the principles discussed in this paper merit the attention of architects, engineers and contractors.

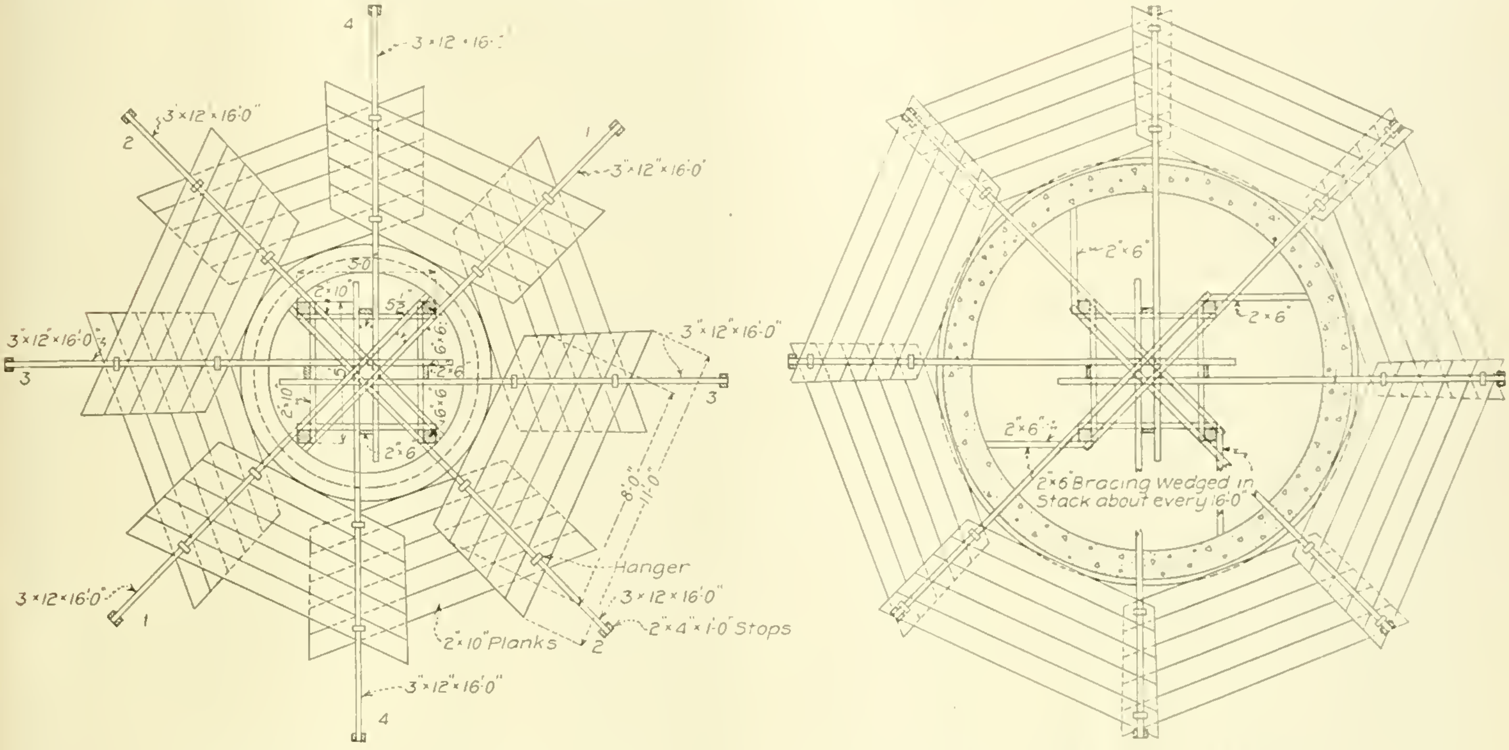
The custom of asking an number of bidders each to find out the quantities of labor and material required in the erection of a building (in an impossibly short time) and then to estimate the cost of these items and make a lump-sum bid for the whole work is absurd. The manufacturer schedules his requirements and takes bids upon that schedule and usually lets the contract for his supplies on unit prices. But when it comes to building a factory, a house, a theater or a church, the contractor is invited not only to guess or calculate how much it will cost before it is built, but also to guess or calculate how much material goes into it; and this work is done not by one builder, but by eight or a dozen, and the labor of all except one is absolutely wasted. It is hard to impress either the architect or the building owner that this waste is unnecessary or that it can be eliminated. The building owner, if he is engaged in manufacturing, would say it is only selling expense, anyway, and selling expense has to be incured. But the selling expense of a builder's office is a very different thing from a manufacturer's selling expense. The manufacturer's salesman has to draw the attention of the prospective purchaser to the merits of the goods he is offering and bring himself before the purchaser's notice. But in the case of the contractor, when all this work has been done, a very heavy expense is incurred by the builder in figuring out how much to offer to sell his work to the building owner for. In the case of the salesman all this is done for him before he starts out upon the road, and if every salesman had to figure out the cost on the spot of the articles he was furnishing, and also the quantities of articles he would be required to furnish, his duties would be very much more arduous. The cost of an automobile or a piano is known before it is offered for sale, but the contractor does not know the cost of his building until it is built. It is difficult to impress upon the architects and owners the amount of time that is required to make an accurate schedule of quantities. If a builder wants to take off a $100,000 job properly, it requires at least a week of hard work day and night. So much of the builder's work is unproductive that he does not put in as much time as he should, or would like to, on the estimate for any one building. Owing to the short time that he is allowed he must take the quantities off very roughly. During the day material men and sub-contractors are busy with his plans and he has only the night. Many contractors have to be satisfied if their estimates of quantities are within 5 or even 8 per cent of the accurate ones, as they only take them off roughly with very little detail. Those contractors who wish to estimate accurately and closely have to spend much time on their estimates. It is disheartening to figure week after week and get very little of the work on which so much time and thought has been given.

This matter was not so serious when competition was not so keen and when estimates were not made so closely. The estimates of many of the best contractors then were the roughest of guesses. They carried a considerable margin for contingencies. The old books on estimating tell the estimator to add from 5 to 10 per cent to the estimate to cover contingencies. As a consequence estimating was not a very serious affair and contractors were very prosperous people. At the present time estimates have to be figured very closely; to figure them closely they have to be figured accurately, and the time and expense of doing this is very great. Many old-time builders who are still with us, and who estimate by rough appropriations, are losing their hold on the business situation. In my own city the large builders whose names were names to conjure with 15 or 20 years ago are today bemoaning their lack of work. It is all being gained by the younger men who are figuring much more closely than they ever dreamed of figuring or knew how to figure.

DIFFICULTIES OF THE CONTRACTOR'S ESTIMATOR.

It is assumed by the architect and the building owner that when his plans and specifications are complete the quantities of labor and material required for the erection of a building can be accurately and correctly figured by the contractor's estimator, so a lump-sum bid is asked for and the plans are given to the contractors to figure. The fallacy of this assumption lies in the fact that it is not possible for a contractor's estimator, however expert he may be, to understand and read the plans of the architect or engineer in the short time allowed. It stands to reason that a contractor's estimator cannot in one week become as familiar with the plans as the architect or engineer who has spent several months in creating them.

When the plans and specifications are completed it is obvious that certain quantities (unknown) of labor and material will be required to complete the building. No amount of careful figuring by any contractor can decrease or increase these quantities in any particular. They are fixed by the plans and specifications. When ten contractors figure, each one competes with the others in seeing how much or how little brickwork, concrete, plastering, etc., he can figure in his estimate and still feel that the job is safe. None of them will get exactly the same quantities in their estimates. The contractor first must compete in accurately figuring quantities before he starts to put a price upon his quantities and thus starts the price competition. No figuring will make one brick take the place of two, or will make 100 bbls. of cement mix 1,000 cu. yds. of concrete. These quantities are fixed and it seems absurd that there should be this unnecessary competition in quantities before prices are put on estimates.



(a) Scaffold in Highest Position (b) Scaffold in Lowest Position

Fig. 3. Plans of Scaffold and Outriggers Showing Same in Highest and Lowest Positions for Wrecking Stack.

In 1908, the Illinois chapter of the American Institute of Architects prepared a report which deals with this subject. Their report states that they investigated the office work of five representative contractors. They found that the average number of contracts estimated by each builder in one year was 72, and that the average number of jobs which he obtained from these estimates was 10. The average value of the time spent by the office force of each firm on each estimate was $125. The everage *expense* on each estimate, including additions by sub-contractors and material men, was $378, making a total of $503 spent by each builder on each estimate. In addition to these expenses the head of each contracting concern spent 21 per cent of his total time in estimating or in overseeing the work of estimating. The report also points out that the expense of six bids on a building costing between $100,000 and $150,000 was over $3,000, which was between 2 and 3 per cent of the entire cost of the work. It is further estimated that in Chicago the contractors spend in one year $1,250,000 in the preparation of estimates, of which only $210,000 was productive expense resulting in getting estimates.

It seems obvious that in the long run this unproductive sum must be paid by the ultimate consumer, who sooner or later must pay the bill, probably without knowing it. Directly or indirectly, it is shifted by the building trade or building material trade to the building owner, together with the bad debts caused by contractors' failures, more than half of which are due to estimating on incorrect quantities.

The skeptical will think that the writer is presenting an overdrawn picture of the inaccuracies of builders' estimate and the resultant losses. Ask any credit man what the worst risks are for credit; builders and contractors will be among the number. The percentage of failures is very high, and a very large proportion of these failures is due, not to mismanagement, but to bad estimating, and of those due to mismanagement much of the loss is due to the time spent on estimating, which should have been given to the supervision of work in progress. It is unfortunate that the building industry, handling as it does such a large proportion of the trade of the country (the whole of the output of the cement, burn-clay and cut-stone trades, and a large part of the output of the steel, glass, paint, lumber and slate trades) should be thus handicapped.

SCOPE OF THE QUANTITY SYSTEM OF ESTIMATING.

Is there any way in which the existing state of affairs can be improved? The writer believes there is, and to answer this question he offers the following discussion of the "Quantity System of Estimating": This system has been much discussed, but its importance merits full consideration. In order that there will be no misconception it will be well to define what he understands by the "Quantity System of Estimating." In brief, a "Bill of Quantities" is an accurate schedule of the quantities and description of the work (labor and material) required in the erection of a building, in accordance with drawings and specifications supplied by the architect or engineer

Fig. 4. Vertical Rods and Wire Mesh After Concrete Was Removed—Rod Shown Bent Provided Exit from Tower to Scaffold.

A "Quantity Surveyor" is an independent party appointed by the owner, architect, or engineer, whose duty it is to prepare the bill of quantities at the owner's expense.

Under the "Quantity System" (as advocated by the "American Institute of Quantity Surveyors") bills of quantities will be furnished free of expense with the drawings and specifications to all bidders desirous of bidding in competition. Lump-sum bids will be based upon the quantities of work scheduled in the bill of quantities, the character of same being ascertained from the plans and specifications. In the event of any errors, the cost is adjusted between the owner and the builder by the surveyor, so that the builder takes no liability for the accuracy of the quantities. This is a matter for the owner and the surveyor to settle between themselves, and this will be discussed later.

THE QUANTITY SURVEYOR AND HIS DUTIES.

The quantity surveyor should be a man who makes it his sole business to take off quantities. He should receive the plans from the architect and compute from them the quantities of material and labor for the building, in the way that the contractor's estimator does now, but as he gives his whole time to the work he is able to work more carefully and has more time to make a careful and detailed estimate. Moreover, if he is paid a proper fee for his work, he will be able to have all his figures carefully checked.

The quantity surveyor should take off the whole job, and should not confine himself to those trades which will be executed by the general contractor's own force. A bill of quantities should contain separate schedules for mason work, structural steel, earthwork, concrete, structural carpenter work, finished carpentry, hardware, plastering, plumbing, painting, ornamental ironwork, etc.; and the quantity surveyor should be a man of sufficient technical education and training to know enough about all these trades to be able to take off quantities for them in the manner in which various contractors and sub-contractors measure them. It should be the aim of the quantity surveyor to take off and describe the items in the most serviceable manner to the contractor, and to give such information as will make the schedule entirely clear. This is a very important matter and upon it largely depends the success of the quantity surveying movement. We all know that similar work executed under different conditions on the same job will vary in cost. Brickwork in manholes will cost less to build than walls of office buildings, likewise 8-in. brick walls cost more per thousand than 12-in. walls. Concrete formwork is an item of expense which bears no relation to the yardage, and must be measured separately. Every surveyor should work on the same method,, measuring all work net, deducting openings in brickwork, carpenter work and plastering, etc., measuring forms separately from concrete work, and in general giving items in detail.

If the quantity surveyor gives his whole time to this work it will be much easier for him, by consultation with the architect, to straighten out difficulties and points which are not clear than it is for the estimator, and he should be able to discover and point out such inaccuracies in the plans as do occur even in the plans of the most careful architects. Moreover, he should be able to suggest amendments to the specifications where they are deficient in description or are ambiguous in meaning. He is often able to start work before the plans and specifications are completed and can thus save time. When he has finished his "taking off" the items of his estimate are collected and sorted and brought into schedule form. A separate schedule or bill is made out for each trade; these are bound together, and a copy is made and sent out to the general contractors who are invited to tender. The schedules contain sufficient description as to enable the contractor to understand fully the items which he is asked to price; therefore it is not necessary for him to spend any considerable time in going over the plans and specifications. When a contractor receives the schedule of quantities he prices all of the items in the trades which he proposes to execute, or which he is competent to price. He then passes to the sub-contractors the schedule of items which he intends to sublet. After the pricing is completed and extended the totals of the various schedules are collected, and such items as profit and incidentals, insurance, etc., are added.

ADVANTAGES AND DISADVANTAGES OF THE QUANTITY SYSTEM.

There are, of course, disadvantages as well as advantages in the quantity system. One great advantage is that all contractors *are bidding on the same basis*. It frequently happens that with the greatest of care there are items on the plan which are not easily understood, and statements in the specifications which are indefinite or ambiguous. In such cases it is the man who has figured on the cheapest manner of doing the work who gets the job. Later on when he finds that his guess is wrong trouble at once arises. As the quantity surveyor has been in much closer touch with the architects than it is possible for the contractors' estimators to be, he is able to have corrected these indefinite points before his quantities are finished. Moreover, since the contractors are competing *on prices only and not on quantities*, the owner is better assured that he is giving his contract to the man who is able to do his work properly at the lowest possible remunerative price. Under our present system the odds are in favor of the contractor who has the cleverest estimator, regardless of whether his building work is good or poor. There is also less chance of error due to the omission of some important item in the estimate.

It is often urged that quantity surveying will add to the number of worthless competitors, while others have claimed that it will eliminate such competitors. It is not certain that it will do either; but it will reduce the risk of work being let for less than cost to a man who has omitted items.

It has been objected that the adoption of the quantity system would not permit a contractor to exercise the right of judgment in making up his prices. This objection arises from a misunderstanding of the meaning of a bill of quantities. Such a bill would schedule, for example, the required number of fire doors with their description, giving the number of swing doors and of sliding doors and the size of each. But this schedule would not prevent the fire-door contractors from examing the plans and determining the number in the first story and in the pent-houses, etc., and noting difficult or unusual conditions. In a similar manner the schedule of quantities for plastering work would describe the different kinds of plastering. All ceiling plastering, in two coats, requiring ordinary staging, would be classed together, but plastering for a high studded room requiring a specially built stage would be put down as a separate item. Plastering on flat wall surfaces, if all three-coat work, would be made in one item. Plastering on metal lath would be separated, and plastering in narrow widths or on circular walls would also be measured as separate items. The bill of quantities should contain an intelligent description of each class of work and every variety of item of work in that class.

It has been claimed that quantity surveying will reduce the cost of building work. This is doubtful, although logically it should in many cases. It should, however, reduce costs in three ways: First, it is certain that the very large overhead unproductive expense which has been mentioned will be saved; second, the men who formerly spent much of their time in figuring quantities will be free to devote themselves to effecting economies in the conduct of building operations, especially the contractor, who will have more time to supervise his work; and third, it will not be necessary to add any contingent items to the bids to cover possible errors in estimating. On the other hand, the very low bid put in by the inexperienced or careless contractor is likely to be eliminated. It is fairly certain, however, that bids will be *closer* than at present, and the writer believes that the bids of the conservative contractor will be somewhat lower.

Mr. G. A. Wright has well said that an architect or engineer who knowingly advises an owner to accept a bid which is lower than a job can be done for is "evading a moral issue." In addition to this he is a fool if he thinks he can get something for nothing. No amount of inspection will make a cheap building good or compel a builder who is endeavoring to save money at the expense of the job to put in all that is called for. The owner gets what he pays for. Unfortunately, he often does not realize until too late that cheap construction means costly maintenance.

Even if the adoption of the quantity surveying system did not result in lower bids the writer believes it would lower the bids of the more careful contractors, even though it did not raise those of the careless contractors. The owner would have the satisfaction of knowing that he was paying a fair price for his building, and that he was more likely to get a fair building for his price.

The subject seems to have been obscured by the suggestion that a reorganization of or a radical improvement in the methods of preparing plans and specifications in architects' offices is a necessary preliminary step to the establishment of the quantity surveying system. This is not the case. The quantity surveyor's duty is to take plans and specifications as he finds them, and from them compute the quantities of labor and material in the building. It is of course desirable that the plans and specifications should be complete—as the better the plans the better his survey—but even with a poor set of plans he is likely (because of his greater experience and the better facilities at his disposal) to make a better survey than half a dozen competing bidders. But there are no grounds for the statement that quantity surveying would of itself improve any architect's plans or require better plans than the contractor now gets. No quantity surveyor would guarantee absolute accuracy from inaccurate plans. In the latter case the contract should be let on the basis of quantities as a unit price job.

It has been objected that those who have special ability in scaling quantities will lose their advantage compared with those who have not this ability. Yes, and those who learn arithmetic at school also have an advantage over those who have no education. How many men have such skill in scaling quantities that they can view with complacency the continuance of present conditions and are willing to waste time on fruitless estimating? There are very few who feel that they have a decided advantage in this—an advantage such that they are unwilling to forego it. Frankly, just how much of an advantage is it? Does it get them a large proportion of jobs, or is it not a fact that, owing to their skill in scaling and to their care, their estimates are *higher* than those of their less fortunate rivals who discover too late that they have "left out something"? True competition lies in judgment as to pricing, and the real contest—even today—among the better class of estimators is and should be the ability to foretell what a difficult piece of work will cost and how it should be performed.

The architect and owner may think that "Brown" and "Jones" know exactly what each item in the building will cost, and if Brown bids lower than Jones it is because he can work cheaper. This is often far from being true; in fact, the reverse is often the case, the lower bid simply meaning that the contractor who submitted it did not know either his quantities or his costs. The writer is often asked how many of the unit prices used by him in his estimates prove to be accurate, and some are incredulous when informed that none are accurate. The investigator can only hope to get somewhere near the truth; the most successful estimator being the one who most closely approximates the facts.

Again, it is claimed that all contractors will be placed on the same basis and that incompetent ones will be able to secure the bills of quantities which they could not draw off. There are two types of incompetents: a man may be a good calculator but a poor builder; or he may be a good builder but not expert at figuring quantities. The first type of contractor is already figuring against you, and the architect is the only man who can eliminate him; only good can result from more of the second type entering the field to the exclusion of the former.

It is too much to expect that all troubles and disputes will be eliminated by the adoption of quantity surveying. One class of disputes, however, will be almost abolished—those due to ambiguous clauses in the specifications and indefinite work on the plans. The quantity surveyor will make a careful scrutiny of the plans, and he will be able to clear up many points of doubtful construction, due to his close association with the architect. In any case his interpretation of the plans is accepted by the builder and is guaranteed by the owner, and if it is found that there is any error the builder does not suffer.

CONCERNING THE ADOPTION OF THE SYSTEM.

The quantity surveyor is a very important factor in this system and it will be of interest to determine how we are to secure competent men for this work. Shall the system be adopted at once or shall it be a slow development?

The logical men for our first quantity surveyors are the chief estimators of our well-known building contractors. These men could find capable assistants from the ranks of engineering and architectural draftsmen, especially the former, part of whose early training comprises the acquisition of a sound knowledge of the principles of construction.

The manner of making a start will vary so considerably in different cities that the writer wishes to present three methods of procedure which may be used under various conditions.

(a) If the owner of a prospective building is convinced of the advantages of the quantity system the simplest procedure would be for the architect to appoint a man whom he knows is qualified to act, or to ask the secretary of the local builders' exchange to appoint a suitable man for him.

(b) If the owner or architect is still opposed to the quantity system, there is no reason why it should not be put in operation without his assistance, if the builders of an exchange have proper confidence in each other. They could meet and agree that one of their number should prepare a schedule of quantities for the work. If this man was not one of those who were bidding on the job, it would be an added advantage. He could furnish the copies of his schedule to the competing bidders, and they could price them out and bid upon them. In this case payment would have to be made to the man who had prepared the quantity schedule. This could be accomplished, either by having each bidder pay his proportion of the cost, or by having each bidder add the entire cost of the schedule to his estimates, which would throw the entire cost on the successful bidder. The owner would gain little advantage from such a proceeding except a possible saving due to closer bids. The writer does not consider that there is anything improper in this procedure. The cost of estimating in this manner would not be as great as the present cost of estimating, and there is no more impropriety in such a proceeding than there is an a number of builders taking bids from the same material man or sub-contractor. The fact that they have paid a fee for the estimate does not render the transaction improper. The writer sometimes pays fees to sub-contractors for the purpose of getting careful estimates from them on work, so that he may feel free to sub-let it as he desires if his firm secures the job. The costs of such fees are always added to the writer's estimate, although of course they are paid by his company in case the contract is not secured.

(c) A third method which has come to the writer's notice commends itself very highly to him. In Milwaukee, Wis., they are organizing a "Bureau of Quantity Surveying." This is subscribed to by the chief contracting firms in the local builders' exchange. The organization will be made up of the chief estimators in the city, who will receive fixed salaries for their work. Each member of the exchange will subscribe a sustaining amount to the bureau, and in addition will pay a small fee for the use of each bill of quantities. He will also add to the amount of each bid which he makes on quantities furnished by the bureau a percentage, varying from one to two, and if he is successful in getting the job he will pay this fee to the bureau, unless the owner pays it direct. At the end of the year any surplus will be divided among the contributing builders.

In the second and third cases, it is contemplated that the builders themselves will stand responsible for the payment of the quantity surveyor. The writer is not at all sure that it will be possible so to convince architects and owners of the value of quantity system that they will be willing at first to pay for making out the schedules. It is therefore advisable for the builders to install the system, as was done in England many years ago. In that country it was a long time before architects and owners realized that there was any advantage to them in the use of the quantity system. When they finally did realize its value they took charge of the appointment of quantity surveyors, and the common law established their liability for fees. If builders want quantities furnished it is the writer's belief that they will have to show the way by first furnishing the quantities themselves; the owner's co-operation will come later.

If it is necessary at first for the builders to bear the direct expense of a quantity system, it has been suggested that the public will think there is collusion among the bidders. It is certain that in some places there will be suspicion as to the propriety of this procedure. But it does not follow that, because there is co-operation, there is also collusion between builders. Co-operation is doing so much in other lines of business in spite of the suspicions of the public that it ought to be allowed here.

FEES.

What fee ought to be charged for the services of a quantiy surveyor? It is not easy to set a fee which will apply in every case. In England, the mixmum fee is usually a percentage of the lowest bid, and ranges from 2½ per cent on dwellings, churches, and other structures which contain a very large amount of detail to as low as 1 per cent on large, plain, factory and warehouse work. Although at first thought a fee of 1 per cent on a $100,000 factory will seem large, the cost of an estimate on such a factory at the present time is more than this (as has been pointed out), although the cost is spread among the bidders instead of being concentrated in one payment to one party. The only estimating expense incurred by the builder under the quantity system will be for the time spent on pricing the estimate, visiting the site, and making inquiries from material men. This will take much less time than any attempt to scale the quantities, so that the contractor's expense of estimating will be practically negligible. It is very important that proper fees should be charged from the start. If fees are established at the start which are too low to pay for painstaking, accurate work, the work will not be done carefully. It will be extremely difficult to raise the standards, and the whole movement will suffer.

RESPONSIBILITY FOR MISTAKES.

Those who introduce the quantity system will undoubtedly encounter many difficulties. The first point that is sure to be raised is "What will happen if a mistake is found in the quantities?" It is practically impossible to make a perfect estimate of quantities, although the writer has found that most of the estimates made by quantity surveyors were surprisingly accurate. Yet errors of considerable amounts have been found in bills of quantities, and the question naturally arises "How are they to be adjusted?" It is obvious that, if the owner has paid for the bill of quantities, and has guaranteed to the contractor that the bill is correct, any error in quantity or description should not fall upon the contractor, who has used the quantities in good faith. It then rests between the owner and the quantity surveyor as to who shall pay for errors. Some will argue that the owner should pay; others that the surveyor should pay for his errors.

Under the present system the owner does not call upon the architect to pay for mistakes made in the design of a building or for the omission of items from his specifications, the architect being engaged by him in a professional capacity to render service, and so long as he renders that service to the best of his ability he is not held liable for errors. Neither does the owner require a guarantee that the building will not fall down through faulty design. Only in the case of gross, culpable or criminal negligence will the courts hold him liable for errors, and if an architect has forgotten to specify the painting, or through some error figures the over-all dimension of his building incorrectly, the cost of rectifying such errors falls upon the owner. By analogy, it may be argued that the surveyor's contract of service should be of the same character.

On the other hand, if the quantity surveyor is to justify the existence at all, the owner will expect him to furnish some guarantee as to the accuracy of his work. The writer does not see any real objection to his doing this, although the men who are called upon to take off quantities may at first look upon such a requirement with some fear. However, they will find that, under the different conditions under which they will be expected to work, with sufficient time in which to do the work and with proper opportunities for studying the plans with the architects, their work will be more carefully done and will be more accurate. Then, if a proper fee is paid for the quantities, they minimize the liability of error. In English offices, every operation is checked before the next one is made, and mistakes are surprisingly few. In a general way, English quantities are supposed to be correct within 0.5 per cent, and the writer believes that they are. It often happens, however, that one item is slightly in excesse and another is too small (such as different kinds of flooring in the same building), but it is not often that the totals are incorrect. The Quantity Surveyors' Association of England makes it a requirement or rule of membership that every member shall guarantee that his quantities are accurate and shall refund to the owner any expense to which he is put by reason of inaccurate quantities, provided, of course, that correct and properly drawn plans were furnished him before he started to take off the quantities. In an article by a past president of the "Quantity Surveyors' Association," in the American Architect for March, 1914, it is stated that so far it has not been necessary to call upon a single member to make good under this guarantee, which is sufficient evidence of the possibility of making accurate estimates of quantities from proper plans. The writer would have no fear in guaranteeing that his work would be correct, and he believes that others would soon feel sure enough of themselves to do so. It may be objected by the owner that the guarantor is not a man of sufficient financial standing to make his guarantee of sufficient value. To meet this difficulty the writer is not able to suggest a solution, except in the case of the bureau already mentioned. The question of accuracy and guarantee, however, does not seem to the writer to be a very important one, owing to the very few cases in which it is actually needed in settlement. It is one that will take care of itself. If a quantity surveyor has to depend for his professional reputation and standing upon the accuracy of his work there is no greater probability of his doing careless and inaccurate work that there is of the architect or engineer whose standing depends on the quality or permanence of the work he designs, or of the contractor whose reputation depends on the quality of the workmanship he furnishes.

A DISCUSSION OF DIFFICULTIES WHICH MAY ARISE.

It has been asked, "What will happen if the accuracy of a bill of quantities is questioned, and the surveyor who drew off the quantities is required to check them after the building is completed?" Such cases sometimes occur and the writer has been challenged on quantities which he has prepared. He has always offered to measure up the work in dispute on this condition that if the party who made the request was found to be in error he should pay the whole cost and a proper fee for the time spent in measuring. Furthermore, the writer would not be content to measure one small item, but would measure all the work relating to the same. For example, if it was claimed that the maple flooring was short in a building floored partly with maple and partly with marble, he would insist on measuring the whole of the flooring and would very likely discover that the quantity of marble flooring was less than the estimate and that of maple flooring more. Cases have occurred where the building was so changed that the whole structure had to be measured up again at the close on account of changes in plans and requirements of the owner. In this case, the cost of measuring should promptly fall upon the owner. If, however, a measurement of a building showed serious errors in a surveyor's work, the cost of making such measurements should certainly fall upon the surveyor, and the cost of adjusting the errors should fall either on the surveyor or upon the owner according to whether the surveyor had made a guarantee to the owner or not.

It may be asked how the surveyor would deal with items which are uncertain in character at the time the quantities were taken off, such as foundations, where it is expected that soil conditions will be bad. In such a case it is usual for the surveyor to take off quantities for whatever work he considers would likely be needed, marking these quantities as provisional, and insert a note in the schedule of quantities that such work is to be measured and valued when executed. After the work has been completed he would inspect the construction, measure the actual work done, and deduct or add the cost of it to the items already taken in the estimate. For this work he would be paid a fee in proportion to that paid on the original quantities.

One feature of the quantity system as used in England would be looked upon with mixed feeling by contractors here, although it would be welcomed by architects, namely, the settlement for extras. It does not necessarily follow that because a bill of quantities has been made the basis for the estimate it must be used as a basis for the settlement of extras, *unless it is so provided in the contract.* A contractor may base his bid upon the quantities furnished, and may refuse to sign a contract that called for them to be used as a basis for extras. However, if he should agree to use the quantities as a basis for extras, one great bone of contention between architects and builders would be removed and better relations would at once be established. The quantity system provides a method for the measurement and valuation of most items of extra or omitted work, for, if the contractor puts the unit price of each item in his bill of quantities and deposits the same with the architect, a definite basis is laid down for placing values on extras, although builders may object to such a proceeding on the grounds that such figures might be used against them. As far as the writer's experience goes (in this country and in England) he has never found that a builder suffered any real damage by letting his prices be known. Prices are no longer a trade secret. The writer has no hesitation in letting anybody know what prices he uses in making an estimate.

Of course the bill of quantities would not provide a basis of settlement for extras which could not be measured, such as the cutting into and alteration of work already built, and for items which could only be charged as day work. In the English system, such items are valued by an examination of the contractor's time sheets and material records.

Some difficulty might arise if nine builders agreed to use the quantities and the tenth refused. In such a case either the architect could refuse to consider the bid of the tenth or the contractors using the system would have to bid under a certain theoretical disadvantage, namely, the disadvantage of carrying the quantity surveyor's fees in their bids. It is, however, probable that the greater accuracy of the quantities and the fact that it would not be necessary to add any profit for contingencies would offset this disadvantage.

The relative positions of sub-contractors and material men would not be materially altered. Each contractor would supply to his sub-contractors copies of those portions of the quantities which related to their work, which would be used as a basis for their estimates. The writer believes, however, that with a little experience and care any general contractor should be able to price sub-contract work himself. In this case he need not call in his sub-contractors until he gets the job—and this will eliminate all the evils of trading sub-bids, which are today such perplexing problems in our cities.

STANDARDIZATION OF MEASUREMENTS.

One of the greatest difficulties ahead of the establishment of quantity surveying is the subject of standardization of measurements. It is an unfortunate fact that in no trade is there any agreement between contractors as to how any items should be measured. Take, for example, doors and windows. There is no method in universal use for measuring a door, or a door frame, or the trim. Most carpenters will look at the plan of an opening and guess the cost of the work. When we come to brickwork, some masons measure the net cubic content of the brick, others will measure all corners double; some will not deduct openings, while others will deduct half openings; some will not deduct stone work, where stone work is embedded in brick, while others will. Having found the number of cubic feet some will take 19 bricks to the cubic foot, others will use 22, 23 and up to 25 bricks. In considering the methods used in measuring concrete, some contractors measure their forms, and others do not, simply adding so much to the price of the cement and aggregate for the forms. Of those who do measure the forms, some measure the number of square feet, while others measure the number of board feet used. The same disagreement is found in the other trades. It will be necessary to determine some definite standards of measurement before quantity surveyors can proceed very far with their work. It will be necessary for the contractors in each city who intend to use the quantity system to establish some definite standards of measurement which all will be willing to adopt. As far as concrete is concerned, this has already been done by the American Concrete Institute, the writer having been a member of the Committee on Standards. These standards are already in use in many parts of the country, and should eventually be adopted in all parts of the country. It is to be hoped that national standards of measurement in other trades will also be established.

In drawing up a standard of measurement, two guiding principles should govern, although in many cases these may conflict with existing practice. The first rule is, *all work should be measured net as fixed in the building,* no allowance being made in the measurements for waste. Then each contractor must add to his bid the sum which he considers necessary to cover the waste—an item which will vary with each contractor.

The second rule is, *material should in no case be measured to pay for extra labor.* For example, if a plumber is measuring 100 ft. of 1-in. pipe in which there are 10 elbows and 3 tees, it is wrong in principle for him to count each elbow as 2 ft. of pipe. If he considers it necessary the elbows, bends and tees can be counted. However, the net number of feet should be measured, and the price should be increased to cover the cost of the bends and the tees. Likewise, the net quantity of brickwork should appear. In this connection, the writer wishes to say that he considers it very bad practice to price brickwork by the thousand brick. The correct unit for measuring and valuing brickwork is the cubic foot, and a quantity surveyor should use this as a basis for measuring brickwork. Each builder can then allow the number of bricks to the cubic foot which he considers necessary.

In conclusion, the writer does not think that the adoption of the quantity system will prove a panacea for all ills or will remove all sources of trouble or dispute, but it will be a real advance and will remove some of the abuses which are current in the building trades. It will be a difficult matter to establish a satisfactory system in this country. In doing so, we should not be bound by the customs of other countries, although we should be guided by those correct principles which they have adopted. The subject of standardization of measurements will be a very important step towards the establishment of a proper system, and there is urgent need for such standards, even under our present methods. This field should command the activities of some of our national associations of engineers, contractors and sub-contractors.

Some Design and Construction Features of the Baltimore and Ohio Terminal Warehouse in New York.

(Staff Article.)

The Baltimore and Ohio Railroad Co. placed in operation, in New York, at the beginning of the year, a reinforced concrete terminal warehouse, which now gives it one of the most modern freight terminals in the country. This structure occupies the block bounded by 25th and 26th Sts., and by 11th and 13th Aves. It is 68 ft. wide by 353 ft. long, and has eight stories for business purposes, with a mezzanine floor for offices and a basement story. Included in the plant are team tracks and paved driveways to facilitate the handling of business through the terminal, while the building is equipped with all modern devices for moving shipments.

The warehouse is of fireproof construction throughout, and its contents are further protected by a sprinkler system which has as its immediate supply a 50,000-gal. gravity tank located above the roof, and a 25,000-gal. supply in pressure tanks located in a concrete penthouse. In addition to these supplies there are five "Siamese" connections at the street level to which the city apparatus can be attached to replenish the sprinkler system. To supplement the sprinkler system there are two hose risers with 75 ft. of hose at each floor level.

Two tracks with a capacity of 18 cars enter the building. The tracks are so constructed that the floors of the cars are approximately at the same level as the first floor of the building, this floor constituting the unloading and trucking platform. The portion of the lot not covered by the warehouse is occupied by the team tracks and driveways, the capacity of this yard being 75 cars. Every car can be reached by the teams, each driveway being paved with granite blocks. Spanning one driveway and a track there is a five-ton traveling gantry crane for the loading and unloading of heavy freight. The construction of the warehouse necessitated changing the entire track layout, as the switching, which is now done at the west end, was formerly accomplished at the east end.

Each floor, including the basement, is served by six electric elevators having a capacity of 10,000 lbs. each. These elevators are located on the street side, so that all merchandise stored in the warehouse can be taken out without interfering with the handling of freight on the first floor. Large pieces, especially automobiles, are handled in cars having end doors. To permit such cars to be unloaded directly into the warehouse an additional track has been provided with its terminus just outside of the end wall, where there is a door as large as the end dimensions of a car. Adjacent to this door there is a large elevator, 10 ft. wide by 17 ft. long, which is used for handling the large pieces.

DESIGN FEATURES.

As the materials to be stored vary considerably in weight the floors were designed accordingly, the floor load varying from 500 lbs. per square foot on the first floor to 150 lbs. per square foot on the top floor. Above the first floor the building is divided by fire walls into three sections, each of which is served by two elevators and an enclosed stairway which also serves as a fire escape. The columns rest on concrete piers resting on wood pile foundations.

The building is of the flat-slab type, and is the first building of this type to be approved by the building department of Manhattan. Bids were taken on three types of concrete construction: (a) beam-and-girder type with exterior curtain walls of concrete; (b) beam-and-girder type with exterior curtain walls of brick; and (c) flat-slab type with exterior curtain walls of concrete. The lowest bid was for the flat-slab type, and the next lowest was for the beam-and-girder type with concrete curtain walls, the increased cost of the latter being approximately 10 per cent. The increased cost of the beam-and-girder

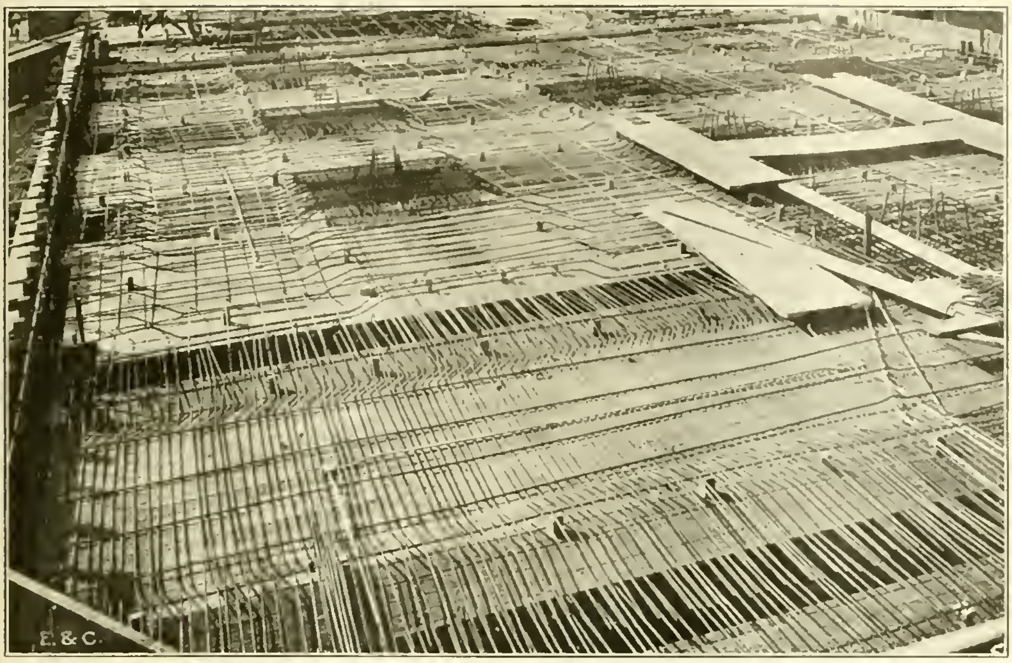


Fig. 1. View Showing Reinforcing Steel in Place for Second Floor of Baltimore and Ohio Terminal Warehouse in New York.

type with brick curtain walls over the flat-slab type with concrete curtain walls was approximately 12 per cent.

As the greatest amount of trucking will be done on the first floor it is finished with a rock mastic wearing surface. The offices on the mezzanine floor have a wood wearing surface, while all storage floors have a granolithic finish.

The building is equipped with electric lights and telephone service, the conduits for which are concealed in the floor and column construction. Articles which require heat are taken care of in two rooms located on the mezzanine floor. There are three toilet rooms located centrally in the building.

The arrangement of the reinforcement for the second floor is shown in Fig. 1. In the foreground are shown the forms for the heavy girders which span the cross-overs of the railroad tracks.

In accordance with the requirements of the building law of Manhattan the curtain walls were designed with double reinforcement, although the walls carry no part of the floor load.

CONSTRUCTION FEATURES.

Before the construction work was started test holes were sunk, which showed that bedrock shelved very rapidly toward the river, the depth to rock at 11th Ave. being about 55 ft. and at the opposite end of the building about 90 ft. For the foundation 3,590 wood piles were driven, ranging in length from 45 to 85 ft. These piles were driven with great difficulty at certain points, as the pile foundations of old piers and an old timber crib bulkhead were encountered. The latter was approximately 35 ft. deep by 18 ft. wide, and was filled with one-man stone. To remove this obstruction it was necessary to drive sheet piling and to remove the stones one at a time. A number of old boats, which had been filled with stone to sink them, were also encountered and were removed after first being enclosed with sheet piling. It took almost a year to get the foundations in shape for the general contractor of the superstructure to start work, whereas the building itself was delivered complete six months after the foundations were completed. Figure 2 is a view of the site, and shows the type of piers used for the columns. It will be noted that the concrete for these piers was spouted into place.

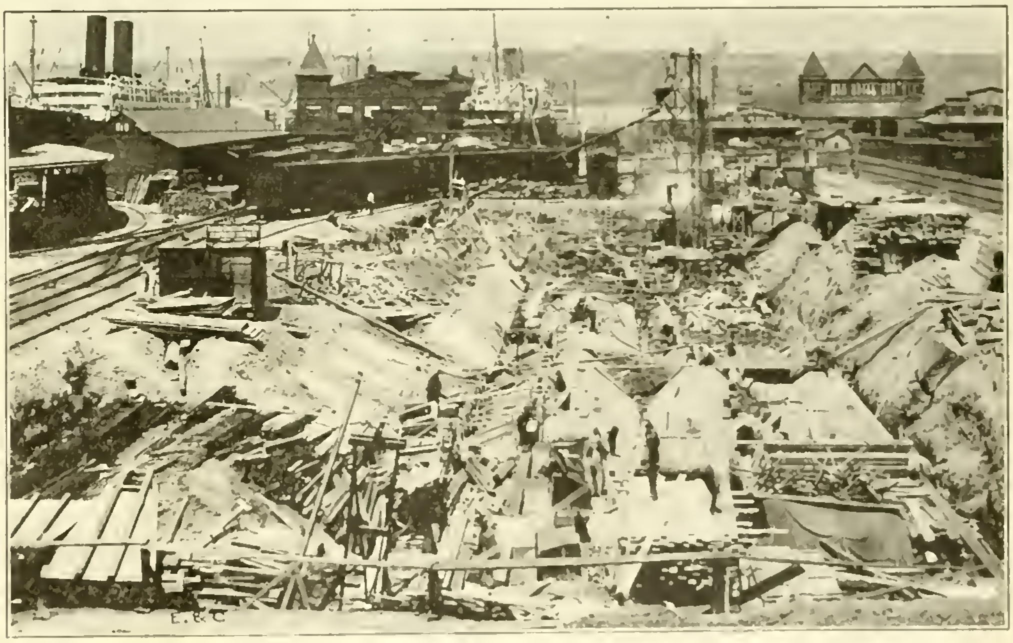


Fig. 2. General View of Site of Baltimore and Ohio Warehouse Showing Concrete Piers and Spouting Tower.

The concrete for the floors was hoisted in a tower and wheeled to its position in the floor. The curtain walls were carried up as monoliths with the floors and columns, instead of being filled in after the latter were in place. Figure 3 shows a portion of the formwork for the exterior walls in place.

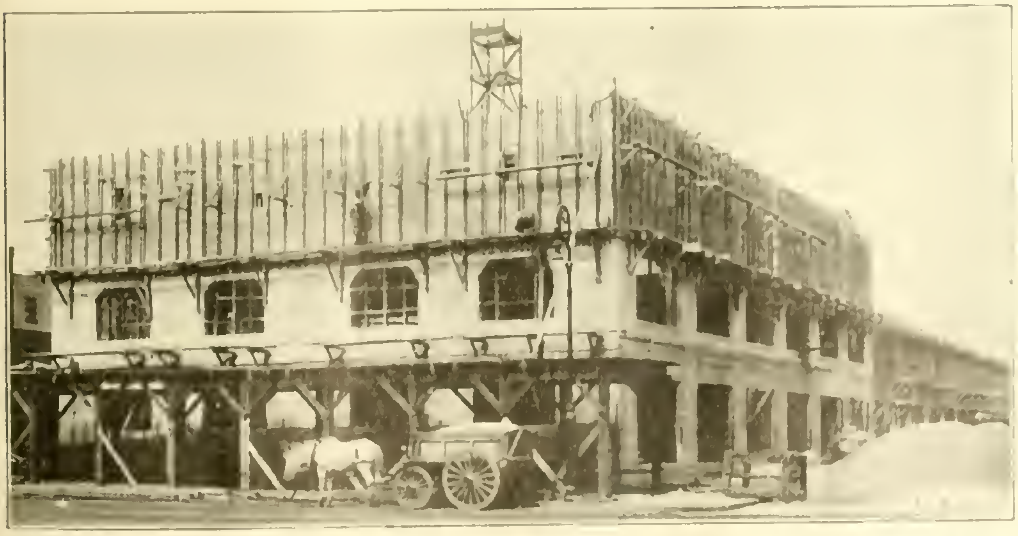


Fig. 3. Forms for Concrete Walls in Place for Baltimore and Ohio Warehouse—Walls and Columns Built Monolithic.

Figure 4 (p. 664) is a view of the building taken after it was practically finished. It will be noted that the architectural treatment is simple, yet effective.

The construction of this building required about 15,000 cu. yds. of concrete and 675 tons of reinforcing steel. The present building covers only about one sixth of the lot area, but it is so arranged and constructed that additions can be built as the business of the company expands.

PERSONNEL.

The new terminal and warehouse were designed by M. A. Long, assistant to the chief engineer, under the direction of Francis Lee Stuart, chief engineer of the Baltimore & Ohio R. R. The field work was in charge of W. B. Redgrave, engineer maintenance of way and M. P. Northam, assistant division engineer.

The Phoenix Construction Co. had the contract for the construction of the substructure, and the Turner Construction Co. built the superstructure. We are indebted to the latter and to the railroad company for the data contained in this article.

Rules for Grading Yellow Pine Timber for Mill Construction.

A new classification has been adopted by the Yellow Pine Manufacturers' Association for yellow pine timber. This classification will come before the convention of the association, which meets in July, and will probably be adopted as it has been approved by the board of directors. A different basis is used for the new classification, as it is based on the strength of the timber, and not entirely on defects. The committee on classification of structural materials, which had charge of the new classification, is composed of M. B. Nelson, chairman; C. E. Slagle, I. H. Fetty, W. J. Haynen, J. W. Martin and J. H. Eddy. The committee was assisted by O. T. Swan, in charge of the Office of Industrial Investigation, U. S. Forest Service, Washington, D. C., John A. Newlin, engineer of tests, U. S. Forest Products Laboratory at Madison, Wis., and A. T. North, consulting engineer for the association.

The new classification is as follows:

The classification of structural material is a question of strength and durability.

In untreated material this can best be determined by its density, as represented by the proportion of summerwood to springwood in the annual growth rings.

Growth rings are composed of bands of hard, dark, resinous summerwood, and lighter, softer, less resinous springwood in each growth ring, and the greater the proportion of summerwood the greater the strength and durability of the timber.

Dense wood shows on cross-section an average of not less than eight growth rings per inch, measured over the third, fourth and fifth inches, on a radial line from pith to circumference, containing in the greater number of the rings one-quarter or more of summerwood, or may have an average of six or seven rings as above, provided in the greater number of rings one-third or more of the ring is summerwood, or wider ringed material if in the greater number of rings one-half or more of the ring is summerwood as above, and must show a sharp contrast in color between springwood and summerwood.

Defects.—The weakening effect of knots is in proportion to the distortion from a straight line of the grain of the wood and to the degree in which they are ingrown with the adjoining and connecting growth grains; therefore, sound knots only ½ in. in diameter or less may be disregarded in considering either strength or durability.

The location of defects in all stringer forms of beams is of vital importance; therefore, beams whose depths exceed their thickness by 3 ins. or more should be divided for classification purposes by imaginary lines into three volumes, namely: Volume 1 consists of the lower one-fourth of the depth in the middle one-half of the length; volume 2 consists of the upper one-fourth of the depth in the middle one-half of the length; volume 3 consists of the remainder of the beam or stick.

Sapwood is as strong as heartwood, but not so durable, and, therefore, the percentage or proportion of heartwood desired should be expressed in any contract.

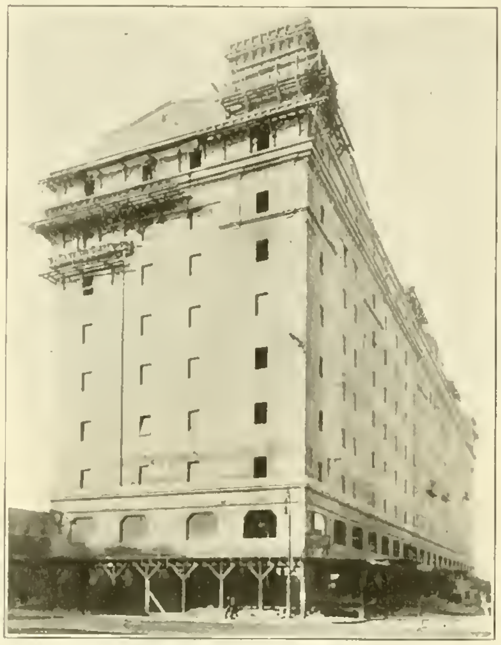


Fig. 4. View of Baltimore and Ohio Warehouse Practically Completed—Note Simplicity of Architectural Treatment.

In a classification it would be cumbersome to attempt to enumerate all defects; therefore, defects allowed, whether specificed or not, are understood to be equivalent in damaging effect to those mentioned applying to the material under consideration.

Grades.—Structural material shall be classified as select structural and as No. 1 structural —sizes 6x8 ins. and up for timbers and 3x12 ins. and up for joists.

Standard Sizes for Rough Timbers.—Rough timber sawn to standard size shall mean that they shall not be more than ¼ in. scant of count size; for example, a 12x12-in. timber rough shall measure not less than 11¾x11¾ ins.

Standard Dressing on Rough Timbers.—Standard dressing shall mean that ¼ in. shall be allowed for dressing each surface; for example, a 12x12-in. timber after dressing on four sides shall measure 11½x11½ ins.

Select Structural.—All timber shall be sound and sawed to standard sizes, dense, free from such defects as ringshake showing on the faces, injurious cross-grain, unsound knots, and decay.

Stringer forms must not have incased or large sound knots in volume 1, must not have large incased knots in volume 2 or unsound knots in volume 3; beam, post, sill and other forms may have sound knots or hard, firm incased knots the aggregate diameter of which does not exceed the width of the face they are in, but no one knot shall exceed 4 ins. in diameter; stringer forms shall show three-quarters heart at any point on the narrow faces, and post, beam, sill and other forms more nearly square shall show three-quarters heart on all faces at any point.

The measure of knots shall be at right angles with the grain of the knot.

No. 1 Structural.—No. 1 structural shall include timber answering in all respects to select structural, except that a greater proportion of sap or no restriction as to sap will be allowed, making timbers suitable for treatment and distinguishing them from No. 1 common timber.

Irrigation and Drainage Difficulties in Egypt.—Successful growing of cotton and other crops in countries like Egypt where the rainfall is excessive at some periods of the year and wholly lacking at other periods, involves besides irrigation, extensive drainage works.

In the Nile Delta, much of which is nearly at sea level, the problem of drainage is often as difficult and expensive as irrigation, the latter being furnished by gravity from the dams, while drainage water must be pumped from the canals. During flood seasons there is always too much water on the fields and under the surface. Just before and after the floods irrigation is frequently excessive, and the whole result is that the permanent water level in the earth is much too near the surface. This damages the cotton crop as much as a drought, and it is therefore necessary to provide drainage canals and to keep the water in them pumped down to the proper level. While the head to be pumped against is not very great—usually 5 to 10 ft.—the volume of water is so great that large pumping plants are required in several places.

Water Works

Construction Methods Employed in Building the Klondike Pipe Line.

The present article describes the conditions under which the Klondike pipe line was constructed and the construction methods which were employed. Rates of wages are given and, in some cases, the progress figures are recorded. The detailed description of the construction methods given may serve as a guide, or furnish comparative data, to other engineers engaged in laying heavy steel pipe. More specifically the article describes:

(1) Conditions of climate and geography peculiar to the far North and affecting engineering work there. In particular, the perpetually frozen ground is mentioned. (2) Details of construction. (3) The difficulty of getting satisfactory foundations. (4) Points disclosed in the test and operation of the pipe line. The article is a reprint of a paper by W. W. Edwards, Assoc. M. Am. Soc. C. E., to be presented before the society on Sept. 2, 1911, and published in Vol. XL of the Proceedings of the Society, p. 1271-1283, under the title: "The Construction of the Klondike Pipe Line."

The Klondike pipe line was built during 1907 and 1908 by the Yukon Gold Co., at Dawson, Yukon Territory. It is an inverted siphon crossing the Klondike River Valley, and was designed for a capacity of 125 sec.-ft., which figure, however, has been slightly exceeded in operation. The maximum head on the pipe line is about 1,100 ft. The total length of 15,760 ft. is made up of: 2,390 ft. of wood stave pipe, 49 ins. inside diameter; 5,850 ft. of riveted steel pipe, 49 ins. inside diameter; and 7,520 ft. of lap welded steel pipe, 43 ins. inside diameter. The steel pipe ranged in thickness from 3/16 to 11/16 in., and came in lengths of from 15 to 30 ft. It is the largest of several inverted siphons along the line of the Main Ditch System of the Yukon Gold Co., 70 miles in length, which supplies water for hydraulic mining operations on Bonanza Creek, near Dawson.

The location of the pipe line was determined chiefly by two conditions: It was necessary to cross the valley below the junction of Bear Creek, and also to keep above the dredging ground of the Canadian Klondike Mining Co. These two determining features left little room for adjusting the line to fit the ground conditions, but it is probable that the location adopted was as good as could be obtained in any case.

The intake end of the pipe is on a comparatively steep side hill. This hill is of decomposed schist covered by a foot or two of soil which supports a growth of bushes, poplars and a few spruce trees. The flat valley of the Klondike is here about 5,000 ft. wide. It is covered with a blanket of Artic moss from 1 to 2 ft. in depth. Below this there is a layer of frozen "muck," or vegetable mould, which is generally about 5 ft. thick, and below this is generally found gravel. This formation is peculiar to the North, and will warrant a little explanation. The frozen "muck," or "frost," as it is sometimes called, is made up of decomposed roots, moss, tree branches, etc., mixed with some little rock and sand, and the whole mass is frozen solid. The moss blanket on the surface protects it from the summer heat, and whenever this blanket is undisturbed the ground beneath the moss remains frozen the year round. If the moss covering is removed, however, allowing air and water to act on the frozen muck, it soon melts. This condition may be found in almost every river valley in the far north, as well as on the sheltered hill slopes. In places the "frost" has been built up by the seasonal thaw not penetrating fully through the winter's freezing, and extends to a known depth of 200 ft. and unknown further depths. This condition makes the question of foundations quite serious. The "frost" is a good foundation if it is not disturbed, but, if disturbed, the structure placed in it must be frozen in and the surface mossed over again to keep out the air and water absolutely. If this cannot be done, the structure must be carried through the frozen muck to good gravel or bed-rock foundation. Frozen muck was encountered all the way across the Klondike Valley and in spots along the pipe up the sidehill as far as the discharge box.

Only two horizontal angles were made in the pipe line, with the exception of those used in making a short offset around the base of a hill. Manholes, drains, and air-cocks were provided wherever necessary. Cast steel expansion joints were put in every 500 ft. to allow for the expansion and contraction due to the extreme variation in temperature, from 80° F. in summer to —70° F. in winter. One three-span steel bridge, 295 ft. long, was necessary in crossing the Klondike River, and a total of several hundred feet of pile trestle or framed trestle were put in at various places along the line. The aim was to have the pipe supported at least once in every 16 ft. Whenever the conditions warranted it, as for instance, when good foundation was deep, trestle bents were built every 32 ft. and king-post trusses were put in to give the intermediate bearing. In good material, sills were merely laid in the bottom of the pipe trench and on these the pipe was supported. Whenever the foundation was within 3 or 4 ft. of the pipe, crib supports were used; if greater than 4 ft., framed bents were used.

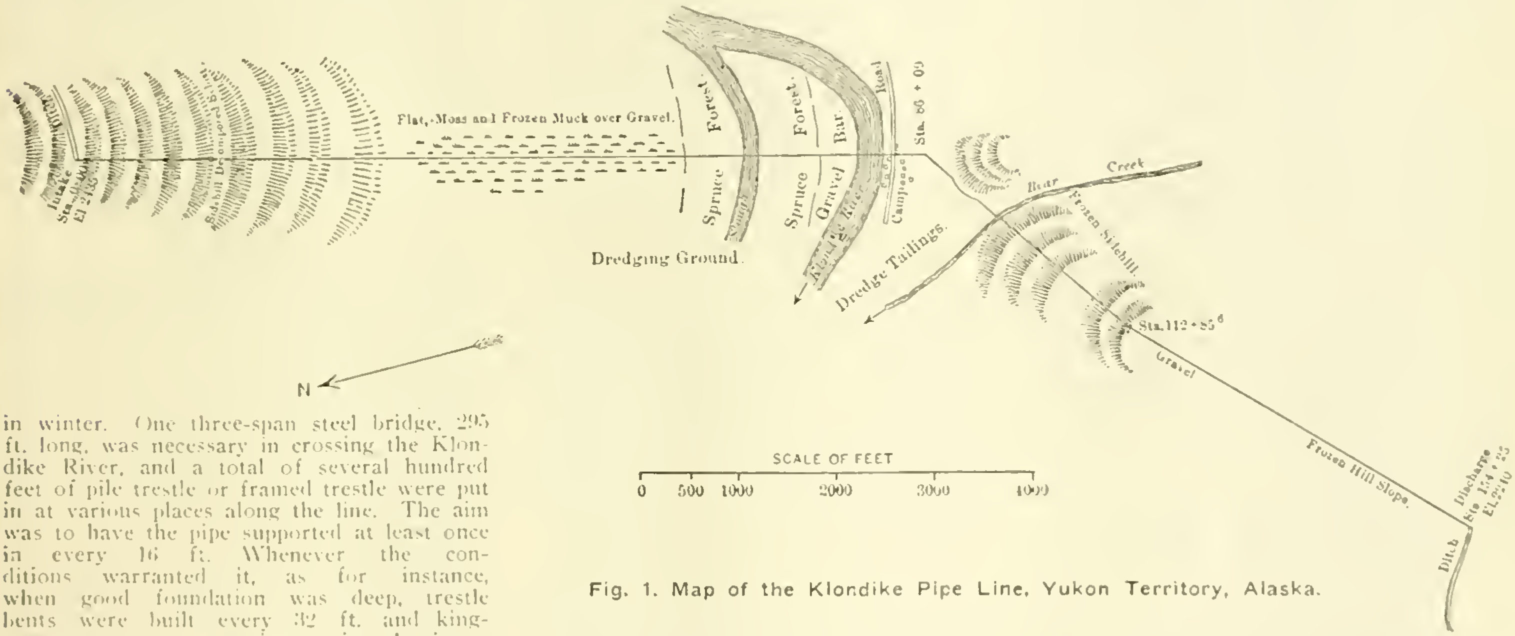


Fig. 1. Map of the Klondike Pipe Line, Yukon Territory, Alaska.

One anchor was put in at least every 500 ft., about midway between expansion joints, and, where necessary, they were used more frequently. Each anchor consisted of a cable wrapped around the pipe in the form of a clove hitch, the ends being made fast to two "deadmen" on opposite sides of the pipe. Each "deadman" was set in the ground about 6 ft. below the surface.

On the side hills drains were dug at frequent intervals to lead off seepage and leakage. At every drain a bulkhead was put in which fitted closely to the pipe and effectually intercepted the water.

In many ways conditions were unique for heavy construction. Common labor was paid $4 per day and board, which meant a total of about $5.50 per day. The working season was short, being limited to five months—May 1 to Oct. 1. Some kinds of equipment were plentiful in the country, notably steam engines and boilers, but in other lines it was sadly lacking. A well selected stock of equipment was ordered for the job, but breakages and unforeseen difficulties sometimes caused awkward situations, in view of the distance from a base of supplies.

Power was furnished from the Yukon Gold Co.'s hydro-electric plant on Little Twelve Mile River, about 50 miles from the pipe line. It was delivered at the main transformer-house at 33,000 volts, and there stepped down to 2,300 volts. From there it was distributed at this voltage to the transformer banks at the several compressors and at the machine shop, where it was stepped down to 220 volts.

Construction was begun in 1907. The right of way was cleared and a small quantity of pipe trench was dug. The three-span steel bridge over the Klondike was erected. Foundation cribs were put in the river bed, the work being done mostly while the river was frozen over and the water low. Piles were driven in the river bottom and sawed off at the gravel surface, and on them the cribs were built. These cribs were then extended to a height of about 5 ft. above high-water level and filled with rock. They were sheathed with sheet iron on the up-stream end to protect them from ice. Derricks were used in assembling the steel spans. During this season, also, a pile approach to the bridge, 400 ft. long, was built, and several other smaller pile trestles and framed trestles. Construction was halted in the midst of the working season in 1907.

In the spring of 1908 construction was begun again. It was desired to have the job completed by the following Oct. 1, the end of the working season. All the pipe was not then on the ground, and what was on hand was not continuous. That is, sections of pipe were missing here and there. As there were so many different kinds of pipe in the line, and as extra pieces were few, it was generally impossible to use sections of pipe interchangeably, therefore work could be started only at certain places. In the course of the work, it was necessary to start laying pipe at four different stations. An error in surveying would lead to awkward consequences where any of these headings joined, as well as at angle points in the pipe line. As there was no heavy machinery or plate for making pipe of this size in the country, and there would not be time to send "outside" for extras, it was necessary to be very careful in surveying.

All the pipe on hand was first measured up carefully and a relation established between actual measurements and those given on the blue prints. An offset line was then run paralleling the located line at a distance of 20 ft. As the seasonal thaw had loosened the original hubs so much that they could not be relied on, measurements were begun from a point on one of the bridge piers. Hubs were driven about 90 ft. apart, and measurements were taken from plumb-bob strings held from tripods. This was done on the side hills as well as the flats. A standard pull of 15 lb. was used the same as had been used in measuring up the pipe on hand. Corrections were made for temperature variations. Levels were run over the hubs, and horizontal distances were computed from the slope measurements and elevation differences. This system resulted in giving a first-class base line, which could be absolutely relied on in setting grades or starting points. Grades were "shot in" by transit. Points were set along the edge of the pipe trench, and were transferred to the bottom, when needed, by using a large wooden square and a carpenter's level. Bench-marks were set every 50 ft. in elevation. Levels were determined by running ahead for half a day and then tying back to the starting point.

In construction, it was frequently found advantageous to depart from the profile in order to avoid excessive excavation, and in these cases grade changes were made by shifting angle points.

Pipe was distributed along the line by a tramway. The cars were pulled by horses across the flat and by hoists on the hills. Of these hoists, two were run by steam, and one by electricity. Signals were given by use of a wire attached to a spring pole. A telephone system also helped in signaling and general communication. Turn-tables were installed on which to change the sections of pipe end for end when necessary. Steel-frame cars were used. In hoisting, a bridle was attached to the end of the pipe, and the cable was shackled to this bridle. When the pipe was at the spot desired, the cars were chained to the rail, and the pipe, with the hoisting cable still attached, was rolled into the ditch. This held the pipe from getting away on the sidehill, and the cable always had so much spring that no harm was done to the engine.

The fitting-up gang used ratchet jacks in fitting up the pipe. The section was first slid down to the last section already connected up and then turned until it was right side up. The "front," or free end, was then raised until the top holes at the back end could be caught by drift pins to the last section. Guide straps were then bolted on at the joint, and the front end was lowered gradually. As the front end came down, the joint was sledged and the guide straps were tightened as necessary. When the joint was entered all around, a ratchet turnbuckle was attached and used to pull it together. Fitting-up bolts were put in every third or fourth rivet hole. Four or five sections of pipe per 10-hour shift was an average for the fitting up gang.

Riveting, caulking, and drilling were done with pneumatic tools. Air was furnished by 8x8-in. Clayton air compressors run by 15-HP 3-phase motors. One air compressor would furnish air enough to operate a riveting hammer, caulking hammer, drills, and air dolly, with not much to spare. The compressor outfits were mounted on platforms and moved from place to place as desired. The riveting of the holes in the bottom of the pipe was done from the inside, and of the upper holes from the outside. Air dollies were used in holding on inside the pipe and spring dollies on the outside. Each riveting gang consisted of a riveter, bucker-up, heater, and passer. An average day's work was from 160 to 180 1-in. rivets, when conditions were good. Fitting-up bolts were used liberally, and the joint was well sledged before driving each rivet. All castings were hand-riveted to the pipe, the riveting being done on the inside.

The expansion joint castings were frequently found to be too large for the pipe. In such cases the pipe was heated by a blast lamp and helled out with hammers to fit the casting. Rivet holes in all castings were drilled in the field after assembling. All caulking was done with rounded-edge fullers, the square tool being dispensed with. The riveted pipe was caulked on the outside only, the lap-welded pipe was caulked both inside and outside. Rivets were inspected with a 1-lb. tapping hammer.

The pipe was coated on the outside with a red mineral linseed oil paint and inside with a quick-drying black paraffin paint.

The wood stave pipe was of California redwood staves thoroughly seasoned. It was laid in a trench 2 ft. deep, and, when completed, was back-filled to a depth of half the diameter of the pipe, so as to give a good bearing.

The intake penstock was equipped with overflow and turn-out, as was also the discharge box. The discharge box was located on a shaded side-hill on frozen clay. This condition made necessary considerable cribbing to protect it from the sloughing of the clay when it thawed consequent on the removal of its moss covering.

Timber was secured from adjoining leases and also from river drives. It was necessary to do considerable work to protect the river banks from undercutting in the vicinity of the bridge, as they were of loose gravel, loam and moss. Piles were driven and shear cribs put in along the river bank above the bridge. Protection cribs were also put in above the pile bents on the bridge to take the brunt of the ice pressure and to break up the ice stream.

Frozen muck, reaching to a depth of 45 ft., was encountered between Stations 84 and 90. Shafts were dug to gravel foundation and trestle bents were put in at intervals of 32 ft. Between Stations 90 and 93, a heavy cut through dredge tailings and solid rock was necessary. This was just at the base of a hill which was covered with frozen muck. The digging of this section was delayed until as late as possible so that the extreme cold would keep the overburden on the slope above from thawing on exposure, and consequent sliding.

A small spot of frozen clay was encountered at about Station 107, and the depth of good foundation being unknown, a corduroy of spruce trees was laid on the frozen clay in the bottom of the pipe trench and this was covered with about 2 ft. of gravel. A bulkhead was built around the pipe above this place to shut off all seepage and leakage, and a drain was run off to the side. The pipe was back-filled and the surface covered with moss again. Apparently, the foundation remained frozen, as no settlement was noticed.

Although work was begun at the camp about April 1, it was not until about May 15 that all the snow was off the ground and it was possible to work to the best advantage. A night shift was put on about May 15 and continued until Oct. 1, by which date the pipe was all laid. The long days of the extreme northern latitude made lights unnecessary on the work until Aug. 1. After that, a system of arc and incandescent lights was installed, which worked very well. The missing sections of pipe began to arrive about July 1, and from that time onward operations were rushed. A force of 350 men was used to push the work to completion. The different headings of pipe were connected up at the expansion joints. This was effected by sliding the outside ring of the expansion joint back on the pipe and then pulling it up to position after the pipe had been lined up.

A tabulation was made showing the space to be left open at the expansion joints at various temperatures, so that they would not close during the hottest or open in the coldest weather. This was followed in setting the joints. Gages were wired on at each expansion joint so that unequal expansion or contraction could be noted and steps taken to prevent it. The coupling bolts for each expansion joint were set so that the lap in each joint could not be less than 1 in. Reference points were also put in on each 500-ft. section of pipe, so that movement could be detected. Anchors were put in at every angle point. These were generally either bulkheads of timber against the pipe, used as thrust anchors, or cables around the pipe attached to "deadmen." When possible, at the angle points, a timber crib was put completely over the pipe and filled with rock. The high cost of transportation and the character of the foundation made concrete anchors out of the question. The only timber available was white spruce, which was soft and brittle, and had to be used with a high factor of safety in all structures.

Some trouble was experienced from the lap-welded pipe. Two makes were used on the job. One, an American make, was perfectly satisfactory, but the other, a German make, was not. One section of the second kind cracked along the weld for a distance of 10 ft. on being dropped into the pipe trench. This was patched and reinforced, and gave no trouble in operation. Another section of this pipe burst in testing under 835 ft. head. Examination showed a flaw in the metal. When it burst, the water was shot out to the side of the pipe line and little damage was done. This was the extent of the trouble with the line in testing.

In operation, the quantity of water flowing through the pipe fluctuated considerably. As the difference in elevation between intake and discharge was 195 ft., this variation caused great fluctuations in the water level in the pipe at the intake end, there being no regulating gate at the discharge end. When little water was flowing, it ran in a very rapid stream down the bottom of the wood stave pipe at the intake end. This had the result of rapidly wearing out the bottom staves, so that considerable patching had to be done.

Bulkheads were put in at each end of the pipe and all other openings were closed as soon as the pipe was thoroughly drained. Thus the air circulation inside the pipe was shut off and the subsequent snow formed a blanket which protected it from the extremes of the winter temperature. It was found that with an air temperature of —30° or —40° F. the temperature inside the pipe was only about —5° F. In operation, the pipe is drained at the close of the working season, about Oct. 1, and opened again about May 1.

The peculiar and therefore the most interesting feature of this work was, as noted before, the study of foundations and the best styles of substructures to be used in the frozen ground. In places in the North, piles of saw-dust have been placed on the moss, and buildings put on this foundation. When water is kept away, this makes a very good foundation, as the saw-dust protects the moss and the frozen ground below from heat and consequent thawing, thus insuring stability. On this same principle, in running a road or railroad grade across frozen ground, it is best not to cut but rather to make fills on top of the moss. Thus, on a railroad grade near Nome, a fill was made of peat cut from the surface of the tundra. When dried out and packed this gave no more trouble—until it burned up—whereas a cut in frozen ground is but an invitation to perpetual trouble from slides and settlement. In flume substructures, on the Yukon Gold Co.'s line, on frozen side-hills, cripple bents were tried and discarded in favor of level mudsills set in the "frost," about 2 or 3 ft. below the surface at the downhill end. These were mossed over and remained frozen unless excessive leakage thawed the ground.

Mr. A. C. Strong, of the Yukon Gold Co., developed a type of structure which suited the foundation very well. He used piles driven into holes previously thawed out by steam points. The steam point is a strong pipe, at one end of which steam is introduced. The opposite end is rounded and a small hole left through which the steam may escape. This point is started into the frozen ground and is pounded down with a hammer as fast as it thaws through the "frost" in its way. These steam points were developed and used to considerable advantage in mining the frozen gravel. In substructure work, the steam point is used until the "frost' is thawed to the desired depth. It is then removed and a pile driven into the thawed-out hole. As the surface moss is disturbed hardly at all in this process, the pile immediately freezes in and remains in this condition. At last reports, pile substructures, such as these, were giving good service. This appears to be the cheapest form of substructure as yet devised for use in ground frozen to any considerable depth.

The foregoing work was done by day labor by the Yukon Gold Co., Mr. O. B. Perry, general manager; C. A. Thomas, M. Am. Soc. C. E., resident manager; H. H. Hall, Assoc. M. Am. Soc. C. E., superintendent of ditch construction, and in general charge of surveys; Mr. C. G. Newton, superintendent of pipe lines; and the writer, division engineer and general foreman.

Design and Construction of the 10-ft. Steel Pipe Conduit, of the Baltimore Water Works, at Loch Raven.

The new Loch Raven dam of the Baltimore water works is being built about a half mille above the old dam. At the west end of the old dam is the old gate-house which controls the flow of water from Loch Raven to Lake Montebello through the Loch Raven-Montebello tunnel, which is approximately seven miles long and is 12 ft. in diameter. For the greater part of its length this old tunnel is dug out of solid rock at a depth in some places as great as 400 ft. below the surface of the ground. In some places where the rock is soft and will not stand this tunnel is lined with brick.

With the exception of the first mile of this tunnel, nearest Loch Raven, the covering over the tunnel is so great that the ground surface is much higher than will be the surface of the water in the proposed high dam with a crest elevation of 237 ft. A. M. T. The tunnel is in excellent condition, and there are only a few places in that portion which is unlined where any rock has fallen from the roof. This unlined portion of the tunnel is seven miles of its length. There is very little leakage except for the first mile where the tunnel for most of the way passes through a limestone under the valley of Mine Bank Run.

On account of the condition of this first mile of the old tunnel after leaving Loch Raven, Messrs. Freeman and Stearns in their "Report on the Enlargement and Improvement of the Baltimore Water Supply," recommended in 1910 that the water from the new dam be conveyed to the old tunnel by a circular tunnel 6,340 ft. long, 12 ft. in diameter, lined with concrete and connecting with the old tunnel at a point about a mile from the present gatehouse. The estimated cost of this tunnel was $480,000.

Mr. Emory Sudler, Division Engineer, in charge of the Gunpowder Supply Improvement Division of the Water Department, made studies of alternate methods for getting the water from the new dam into the old tunnel and it was estimated that for $300,000 a 10-ft. steel pipe conduit could be built from the new dam to connect with the old tunnel just outside of the present gate-house and that the old tunnel could be lined with reinforced concrete for the first mile in such a manner as to withstand the pressure from the water in the higher dam. It is to the design and construction of the 10-ft. steel pipe conduit at Loch Raven that the present article chiefly relates. The information given is taken from a paper by Mr. Ezra B. Whitman in the

Monthly Journal of the Engineers' Club of Baltimore for June, 1914. Mr. Whitman, who is now a member of the firm of Greiner and Whitman, consulting engineers, Baltimore, Md., was Water Engineer and President of the Water Board of Baltimore at the time this conduit was designed and constructed.

The lining of this old tunnel is made possible by the fact that Lakes Montebello and Clifton hold about seven days' supply of water for the city. These lakes can be filled, and then the water can be drained from the tunnel and the work of lining the first mile can be carried on while the city is drawing its supply from the two above mentioned lakes. When the supply from these lakes is exhausted work will be stopped and they will again be filled from Loch Raven. The lining of this old tunnel, however, will not be undertaken until the new dam is completed and the storage of water in it can be used to hasten the filling of Montebello and Clifton between working periods in the tunnel.

There have been great improvements in the building of large steel conduits in the last few years which were introduced in the building of the aqueduct leading the water from the Catskill Mountains to New York City, and Mr. Whitman spent a day with Mr. J. Waldo Smith, Chief Engineer of the New York Board of Water Supply, examining the new, large steel conduits which were being built along the Catskill Aqueduct, and is indebted to him for a great deal of information secured at that time.

The steel used is a soft, ductile, open hearth steel of the character used for flanges and rivets. A cheaper steel might be used, but it is considered desirable to have a high degree of softness and ductility in order that the plates may stretch rather than crack in case of unusual strains due to water-hammer or settlement of the ground, abuse in forcing rivets or to any accident during the construction of the pipe. Old-fashioned wrought iron would be even better than soft steel in resisting rust, but it is more expensive and harder to obtain, and steel can be protected against rust by various means. Not only is steel being used in the great pipe siphons of the New York aqueduct, but it has also been used on the Los Angeles aqueduct and the smaller aqueducts which have been built by Springfield, Mass.; Pittsburgh, Pa.; Portland, Ore., and for pressure pipes on all of the recent great power developments throughout the world.

In steel pipe lines protected with various paints, asphalts and enamels, trouble has been experienced with corrosion, tuberculation and pin-hole leakage. In order to overcome these difficulties the engineers of the New York Board of Water Supply after a number of experiments demonstrating the efficiency of cement mortar in protecting iron from corrosion, developed a method of placing a mortar lining inside the steel conduit and surrounding the pipe on the outside with concrete. The mortar lining also makes it possible to obtain a smooth interior irrespective of projecting rivet heads and edges of plates. So that in addition to promising longer life for the steel conduit the cement mortar lining will give a much larger water-carrying capacity than the rivetted lapped joint pipe which is certain to have its interior surface materially roughened by tuberculation in the course of a few years. Engineers experienced in the water-carrying capacity of steel and concrete pipes have estimated that a pipe 7 ft. in diameter, smoothly lined with cement will, after some years' use carry as much water as an ordinary steel pipe 8 ft. in diameter made in the ordinary way.

The Baltimore steel pipe is constructed of alternate inside and outside cylindrical courses with 10 ft. as the inside diameter of the inside courses. The pipe was made at the shops in sections 15 ft. long with one longitudinal double rivetted lap joint and the circumferential joints were single rivetted lap joints. Changes in line or grade were obtained by cutting and beveling the ends of a sufficient number of courses to produce the desired total deflection or curvature.

Although the pressure on the pipe line would require a thickness of steel of less than ¼ in., yet for structural reasons the thickness was made 7/16 in. In places where thin pipe has been used, great difficulty was experienced in making it keep its shape during construction. A further reason for an increase over the theoretical thickness is that the thicker pipe will last much longer should there be a break at any point in the protective mortar lining. The plates were not painted, but after being inspected at the shop the inside of the pipes were given a coat of white-wash before shipping. No paint was put on the pipes as it was desired to get the best possible contact of concrete and steel.

After the pipe had been rivetted together in the trench it was filled with water delivered from a tank built on the hillside at such an elevation as would give the same pressure in the pipe line as would be obtained by the waters back of the high dam. If any leaks developed under this pressure they were calked until tight. Then with the pipe line still full of water under this pressure the concrete casing was placed around the conduit, the purpose being to hold the pipe in the shape that it will finally assume, so that when the mortar lining is placed no change in shape will occur after the water from the high dam is admitted. It was found that the pipes flattened a good deal under their own and the water load when the pipes were full but with no pressure on them. When the pressure was put on the pipes they tended to return to a circular shape although never quite reaching it.

After the outer casing of concrete was placed the water was drawn from the conduit and the inside mortar coating was placed by means of a "cement gun," the cement and sand being mixed dry and delivered in that condition, by compressed air through a hose or pipe to a specially constructed nozzle where the mixing water was added under pressure and the mixture blown with the compressed air against the surface to be covered until the thickness of 1½ ins. was secured. This mortar lining is reinforced with steel wire netting placed to prevent shrinkage and settlement cracks.

The contractor built a railroad along the entire length of the pipe line and excavated into buckets which were handled from the trench by means of a locomotive crane. This crane was used for taking the pipe sections off the freight cars, which were switched in on the contractors' track, and for lowering them to place in the trench. At Towson Creek and Shanghai Creek the conduit crossed over these streams by means of reinforced concrete bridges.

Previous to laying the pipe concrete cradle blocks were built at intervals so as to come midway between rivetted joints. The tops of these were carefully finished to a surface of a little larger radius than the pipe. The pipe was laid on these cradles and tested with water before being surrounded with concrete.

A few hundred feet below the new dam in the line of the steel pipe conduit, and built as part of it, is a Venturi meter for measuring the flow of water from the new Loch Raven dam to the city. A second Venturi meter will be constructed at the filtration plant, and by means of these two meters it will be possible to detect any serious leakage in the conduit or tunnel.

The contract work was very capably handled by Mason, Hilton & Co., of New York, with Mr. P. A. Beatty, Assistant Division Engineer of the Water Department, directly in charge of the work.

Concrete Piers for Chicago Lake Front.

Residents along the north shore in Chicago have petitioned the War Department to stop the construction of wooden piers along the lake front.

Piers made of concrete and of such artistic design that they will harmonize with the residences along Lake Shore drive and with the architectural character of Lincoln Park buildings are desired.

Classification and Duties of Employes in Bureau of Distribution, Department of Water, San Diego, Cal.

The Department of Water of the city of San Diego, Calif., is subdivided into three bureaus and three divisions, namely, the bureaus of conservation, accountability and distribution, and the clerical and engineering divisions and the division of pueblo lands and forestry. An article published in Engineering and Contracting of May 7, 1913, described the organization of the department, explained the relations of the several subdivisions of the department, and stated the duties of the principal employes in the several subdivisions. The present article relates wholly to the bureau of distribution and classifies all employes under this bureau, states the qualifications requisite for and enumerates the duties connected with the various positions. The matter which follows is taken from the latest annual report of Mr. H. A. Whitney, assistant superintendent and hydraulic engineer, to Maj. Herbert R. Fay, superintendent of the department of water. The bureau of distribution is in the immediate charge of a general foreman and is subdivided into three sections known as the construction, services and operation sections respectively.

GENERAL FOREMAN.

Duties.—The duties of the general foreman are to take all orders issued to the bureau of distribution and segregate and apportion them to the three sections into which his bureau is divided. It is his duty to keep an accurate report sheet showing where each of the several gangs are working, to make out, with the assistance of his section foreman, the charts showing the location of the various gangs for each subsequent day, to make out orders on the superintendent's office for all materials needed to supply the depleted stock, to see that the necessary tools, such as picks, shovels and work boxes are delivered on time to avoid the loss of work in the various departments, to keep account of all orders issued for stock and see that the same is delivered in ample time, and to advise from time to time his various section foremen as to the most efficient methods of pursuing their work. He is also required to perform such other duties as may be directed by his superiors.

SECTION OF CONSTRUCTION.

Foreman.—The foreman has charge of laying all cast-iron water mains and work appurtenant thereto in the City of San Diego. He visits the various gangs working under his direction and advises the general foreman from time to time regarding the progress of the work being done. He sees to it personally that all pipes, valves, fittings and other materials are delivered on the work in time to avoid any delay and to this end it is his duty to inform the general foreman of what material will be needed for each subsequent day.

He makes a daily report for each gang showing the length of ditch, the amount of excavation, the size of pipes, the number of gates and hydrants and the location of same, together with any other work performed under his supervision.

He is also required to perform such other work as may be directed by his superiors.

Sub-Foreman.—The sub-foreman has charge of a pipe laying crew and supervises the laying of all size water mains, the installing of various types of gate valves, fittings and appurtenances adopted by the Department of Water, the installation of branches and fire hydrants or any other work that may arise in connection therewith. He is required to render a daily report setting forth accurately all work performed and material expended by his crew.

Calker.—The calker is required to understand the laying of cast-iron water pipe, how all connections are made, especially the cutting and fitting of pipe, the fitting of all classes of gate valves and specials, how hydrant branches are laid and hydrants set. He is required to know how all size lead joints are run, the proper temperature lead should be heated, how pipe jointers are used, and the manner of calking joints. He is also required to know the names and uses of the different kinds of pipe tools—especially the use of calking tools and the repair of same. He is also required to understand the duties of yarner.

He is next in line for promotion to subforeman, and consequently should have a knowledge of all classes of large pipe laying and possess fitness to handle a pipe laying crew.

Yarner.—The yarner is required to be qualified to determine the qualities of yarn used in pipe joints, the quantities used in all size connections, the different methods used to prepare same for use, how it is placed and set. He must in addition have a fair general knowledge of all classes of pipe laying—especially the duties of calker, and he is required to perform the work of a calker when so directed.

Lead Runner.—The lead runner is required to understand the care and use of lead furnaces, the kind of lead used for jointing, the temperature at which it is heated to be run, the condition it should be in when poured and the manner in which it should be poured. He is required to perform such other work with a pipe laying crew as may be assigned him.

Pipe Layer.—The pipe layer is required to know the different classifications of pipe, the approximate weight of all size lengths of pipe, gate valves and fittings. He is required to know the different methods employed for lowering pipe, valves and fittings into a ditch, how the same are aligned or set for connecting up. He must have a general knowledge of the use of rope knots and hitches, the assembling, setting up and operation of such apparatus as is used in handling pipe by the Department of Water. He must have a knowledge of pipe cutting, the names of tools used and the care of same.

Powderman.—The powderman must have a thorough knowledge of the use of all classes or varieties of blasting powders, the proper quantities and kinds of powder used under all conditions in springing holes or for dislodging certain areas of rock, earth or other material, the different makes and use of caps, fuse and firing batteries. He must also understand the placing and drilling of holes for receiving a charge, the different kinds of hand drills and their use in all classes of material.

Blacksmith.—The blacksmith is qualified to perform ordinary iron work, which includes the sharpening and proper tempering of picks, calking tools and pipe cutters of various makes, and the repair of such equipment as is used by a construction crew in laying cast-iron pipe, or such other work as may be assigned him.

Lantern Tenders—Water Tenders—Laborers.—These men must be physically fit to perform common labor such as is required by the Department of Water, and must be registered voters or taxpayers in the city.

SECTION OF SERVICES.

Foreman.—The foreman has charge of the laying of all water pipes 3 ins. in diameter and smaller, all service lines, meters, meter boxes, meter box covers and any class of pipe fitting work in connection with the service work. He also has charge of all night troubles of any character whatsoever under direction of his superiors, and to this end he is required to call out the proper crew to repair a break or leak in any city water main, service connection, hydrant, or adjust any trouble to the water system requiring his services. He is required to make a daily report of all work performed by his wagon crews.

Pipe Fitter.—The pipe fitter has charge of a wagon crew and all work assigned to be performed by the crew. He is qualified to perform all classes of pipework necessary in installing service lines including the use of the various types of machines used by the department for tapping and connecting up to all size mains; laying service lines and connections; installing safety valves, meters, meter boxes and meter box covers, and in addition he must have a thorough knowledge of all classes of small pipe work done in the department.

Pipe Fitter's Helper.—The helper is required to assist a pipe fitter in the performance of his work, and must be qualified to perform ordinary pipe work, understand the use of the various types of tapping machines used in the department, the classification of small pipes and various fittings, valves and connections. He must be qualified to assume charge of a wagon crew in the absence of a pipe fitter.

Wagonman.—The wagonman must know the classification of pipe, fittings and tools used in laying service lines and the installation of meters, valves and fittings, and must be especially fitted by experience to perform any class of labor in connection with service work.

SECTION OF OPERATION.

Foreman.—The foreman has charge of the machine shop, blacksmith shop, carpenter shops, department stables and storage yard, and all work performed therein is done under his direction. He is charged with the care and preservation of all means of transportation, including wagons, automobiles, auto trucks and motorcycles. All meter repairing and testing is done under his direction. He attends to the delivery of all material and supplies under instructions from the proper authority. He has charge of all reservoirs, stand pipes, pumping plants and pressure regulators and makes recommendations regarding the repair of same. The repair of all faulty gates and re-installation of all hydrants is done under his supervision. He does such other work as may be assigned him.

Machinist.—The machinist must have a thorough knowledge of all classes of machine work and shop management. He must understand the construction, operation and repair of gasoline engines and in addition to directing all work performed in the department machine shop, must perform any other machine work that may be assigned him.

Mechanic.—The mechanic must possess special fitness to do all classes of carpenter or wheelwright work in connection with making the repairs to the department transportation equipment. In general he is qualified to perform any class of repair work he may be called upon to do in the department.

Truck Driver.—The truck driver must be qualified to operate the auto trucks owned by the department, and understand thoroughly their construction and care under all conditions, and be qualified to make minor repairs under emergencies. He must also have a thorough knowledge of the state and municipal laws governing the operation of motor driven vehicles.

Blacksmith.—The blacksmith must be qualified to perform all classes of blacksmith work, especially the sharpening and proper tempering of picks, calking tools and pipe cutters of various makes, or any other class of iron work that may be required in repairing the equipment of the department.

Blacksmith's Helper.—The helper must be qualified to assist the blacksmith in the performance of his work and must possess special aptitude and fitness to prepare himself for the position of blacksmith.

Horseshoer.—The horseshoer must by experience and fitness be qualified to perform all kinds of horseshoeing, understanding thoroughly the care of the hoof and the proper setting of all styles of shoes, and in addition he must have a knowledge of general blacksmithing.

Pump Operator.—The pump operator is required to possess a general knowledge of the care and use of gasoline engines and pumps and especially those in use in the Department of Water. He must possess enough mechanical ability to make minor repairs to the pumping apparatus under his charge.

Meter Repairer.—The meter repairer must understand the construction and operation of all styles of water meters in use by the Department and must possess enough mechanical ability to make all repairs necessary to same. He must also understand the different methods employed to test water meters and means of determining their accuracy.

Apprentice.—The apprentice must possess aptitude and fitness to assist the meter repairer in the performance of his duties with a view to preparing himself for the position of meter repairer.

Teamster.—The teamster is required to understand the care and handling of horses in the city of San Diego and understand the loading and unloading of heavy weights. He must also understand the traffic ordinances of the city.

Pipe Man.—The pipe man must have a knowledge of all size water pipes, valves and fittings and the care and preservation of same.

Cement Man.—The cement man must be qualified to perform all classes of concrete and cement work, and to this end he must understand the construction, setting and removing of forms, the proportion and kinds of materials to use in all mixtures of concrete and cement work. He must understand the quality of materials used in cement work so as to be able to select materials for their efficiency.

Shopman.—The shopman must understand the classification and preservation and care of the different materials and supplies used by the department and must possess clerical ability to keep stock records.

Emergency Man.—The emergency man must possess from experience in the Department of Water ability and fitness to make repairs on account of a break or leak in the system or perform any class of pipe work required to be done in making emergency repairs. He shall have a good knowledge of the distributing system so as to know the proper action to take in case of an accident of any sort requiring discretionary and prompt action.

Stableman—Yardman—Watchman. — These men from experience gained by their employment in the Department of Water must possess special fitness to perform such work as may be assigned them on the care and preservation of department property under their charge.

The Cleaning of Water Service Pipes by Means of Paper Wad Propelled by a Force Pump, Boston, Mass.

Those who are responsible for the operation and maintenance of water distribution systems are frequently advised by water takers that service pipes are partially or completely stopped up. This, naturally, is especially likely to be the case in old systems as is well indicated by the following quotation from the latest annual report of Mr. Frank E. Merrill, Water Commissioner of Somerville, Mass.:

> The number of cases of poor pressure on old water services continues to increase, and cleaning out the pipes and fittings and providing a good flow of water has grown into an important branch of the department work. In fact, the demands for improved service conditions have now become so frequent as to require practically the sole attention of one gang of men, and there appears to be no likelihood of any relief, as the service pipes are continually growing older and in need of attention.

A very interesting topical discussion, before the New England Water Works Association, on the use of the force pump in cleaning service pipes is reported in the Journal of the Association for March, 1914. Mr. George H. Finneran, assistant superintendent of water service in Boston, led the discussion and the major portion of the information here given is based upon his remarks.

Where lead pipes are used as services the stoppage, partial or complete, usually occurs at the corporation cock or tap where rust or corrosion accumulates. This matter comes in from the main and lodges in the cock. The usual method of relieving a situation of this kind is to dig down to the cock and clean it out, and as a rule the results are satisfactory. This method is expensive, however, in cities having expensive pavements, and the excavating operations in the street also interfere with the public convenience.

The Boston water department officials have devised the method here described to supplant the disadvantageous procedure above mentioned.

The high pressure pump used in cleaning services is a lever-handle hand pump. It is a light, compact pump and can easily be carried around by a plumber and his helper. A connection with the service pipe is made just inside the cellar wall, usually at the stop cock. The stop-cock is used to control the water from the street. Before connecting the pump to the supply pipe a wad of paper is inserted in the supply pipe. That is the essence of the operation, the secret of its success. This wad of paper completely fills the interior of the pipe. After inserting the paper wad the pump is connected to the supply pipe. Water is supplied to the force pump by means of an ordinary force pump which draws its supply from a pail. Of course the supply pipe stands full of water at city pressure when the force pump is brought into operation. Therefore any water pumped in by the force pump raises the pressure in the house end of the supply pipe and propels the paper wad towards the street main until it encounters the obstruction. There may be a very small water way through the obstruction, perhaps the size of a pinhole or perhaps larger, but the wad of paper is forced against the rust or corrosion, and completely dams up the little waterway, thus making a complete stoppage. A few more strokes of the pump and sufficient pressure is accumulated to break down the entire obstruction and carry the rust or corrosion and the wad of paper into the main and thoroughly clear the cock.

The removal of stoppages in this way has proved very satisfactory. In one case it required 17 minutes to fill a pail or a vessel that contained 1 cu. ft. of water before cleaning the pipe, and after the clearance the same amount of water was drawn in 23 seconds. The improvement was well worth the effort.

The method is also efficient in clearing out iron pipes inside of buildings. It is desirable to have an open end to the iron pipe. The wad of paper is inserted in the pipe and the pump is applied. As the wad proceeds it scours the walls of the pipe and carries the rust with it. In one case recently 3½ lbs. of iron rust were removed in this manner from 10 ft. of 1-in. pipe.

At first, while experimenting, ordinary newspaper was used for the wad or the pellet. As a general thing it was not very sanitary, so now good quality tissue paper is used. It is compressed very hard, but the moment it gets out into the main it unfolds and dissolves and is carried away somewhere—through a hydrant, perhaps. It is a simple operation, the prime factor of which is the little pellet which for the time being creates a complete stoppage, where prior to the operation there was only a partial stoppage.

The method here described will remove the very hardest mass of corrosion. By this method corporation cocks have been cleared in which the corrosion was so firmly in place that its removal by driving out with a tool would have been necessary if the excavation method had been employed. Naturally the water helps to dissolve and soften the corrosion. As a rule pressures exceeding 150 to 200 lbs. per square inch are not necessary. A gage should be attached to the pump so that the operator will not apply a higher pressure than the service pipe can stand. A length of about 50 ft. is the greatest so far cleaned by this method in Boston.

In case the service pipe changes in diameter at the curb line, with the house end larger than the street end, the wad of paper in moving out toward the main would soften and accommodate itself to the smaller area. The wad becomes plastic after immersion. At first the wad is hard, but the longer it remains in water the softer it becomes, and it accommodates itself to the shape of the pipe and the condition that it meets.

The force pump used in Boston was made by the Blake Pump Co. It was built several years ago for use in the machine shop in testing large gate valves. This pump cost about $50. This pump was used because it was available but the discussion developed the opinion that an ordinary boiler testing pump which throws a very small amount of water but under, perhaps, 200 lbs. pressure would answer the purpose. Such a pump costs about $15.

Method of Remodeling the Underdrain System of the Mechanical Filter Plant at Charleston, S. C.

In April of 1912 the system of underdrains of the mechanical filter plant at Charleston, S. C., began to work poorly. The strainers and laterals became incrusted with a lime deposit. Cleaning the strainers failed to relieve permanently the condition, as in a few weeks they were incrusted as badly as before cleaning. The method employed in overcoming this difficulty is here described and illustrated, our information being taken from a paper by Jesse M. Worthen, M. D., superintendent of the Charleston Light and Water Co., before the Illinois Water Supply Association, published in the annual proceedings of the Association for 1914.

There were three styles of strainers in use, first the mushroom strainer of the New York Continental Jewell Filtration Co., second a small cast bronze strainer with small holes drilled in it, and third a small gauze strainer. The last two were made by the Simplex Valve & Meter Company of Philadelphia.

The incrustation seemed to be a calcium carbonate formation, although no analysis was made of it. The openings in the strainers were filled completely. A heavy deposit was found on the inside as well as on the outside of the strainers, and also on the inside of the laterals.

The raw water is taken from an impounding reservoir of 2,500 acres. The land flooded by the water of the impounding reservoir was not cleaned or stripped. It was composed partly of salt marsh land. The capacity of the impounding reservoir is 3,000,000,000 gals.

Average analyses of the raw and filtered water in parts per million are as follows:

	Raw.	Filtered.
Chlorine	17.0	17.0
Color	150.0	15.0
Turbidity	3.0	0.0
Alkalinity (to erythrosine)	10.0	12.0
Alkalinity (to phenolphthalein)	0.0	0.0
Hardness (soap test)	22.1	36.1
CO_2	7.3	1.1

The treatment of the raw water consists of the addition of two grains of sulphate of alumina as the water leaves the low lift pumps on the way to the sedimentation basins. There are two basins with a capacity of 5,000,000 gals. each. The water is pumped to basin No. 1, from which it flows through a 24 in. pipe into basin No. 2. As the water leaves basin No. 1, 0.25 grains per gallon of sulphate of alumina are added as a recoagulant. From basin No. 2 it flows by gravity to the filters. The period of sedimentation is about 20 hours in each basin, or a total period of sedimentation of 40 hours.

The filter plant consists of twelve wooden units, with a capacity of 500,000 gals. each in 24 hours. To bring up the alkalinity in the filtered water, 0.75 grains per gallon of hydrat-

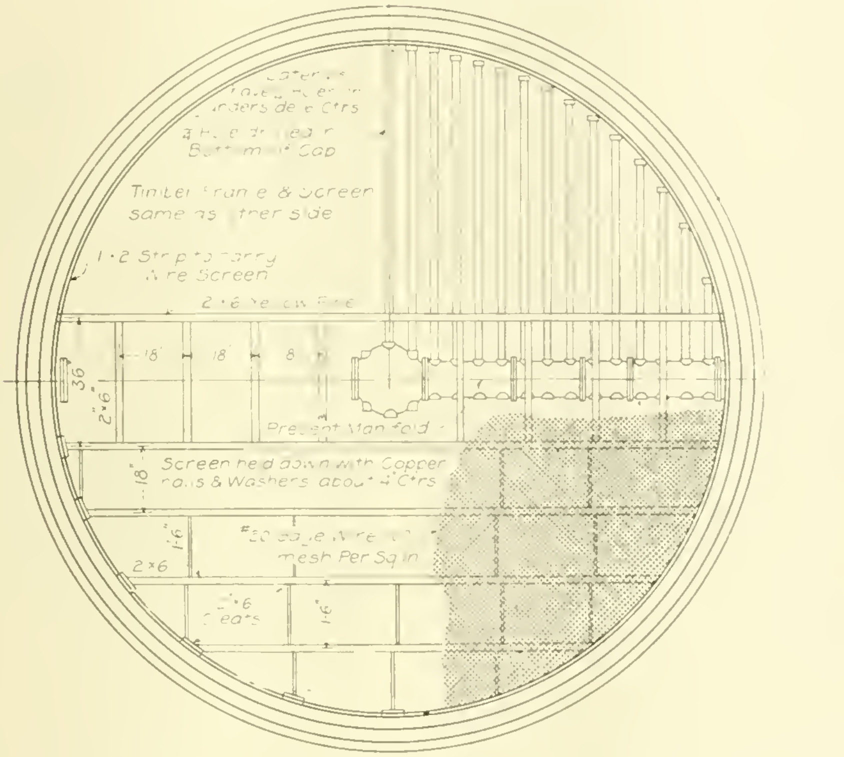


Fig. 1. Plan of Framing for Screen of Remodeled Underdrain System of Charleston, S. C., Mechanical Water Filter.

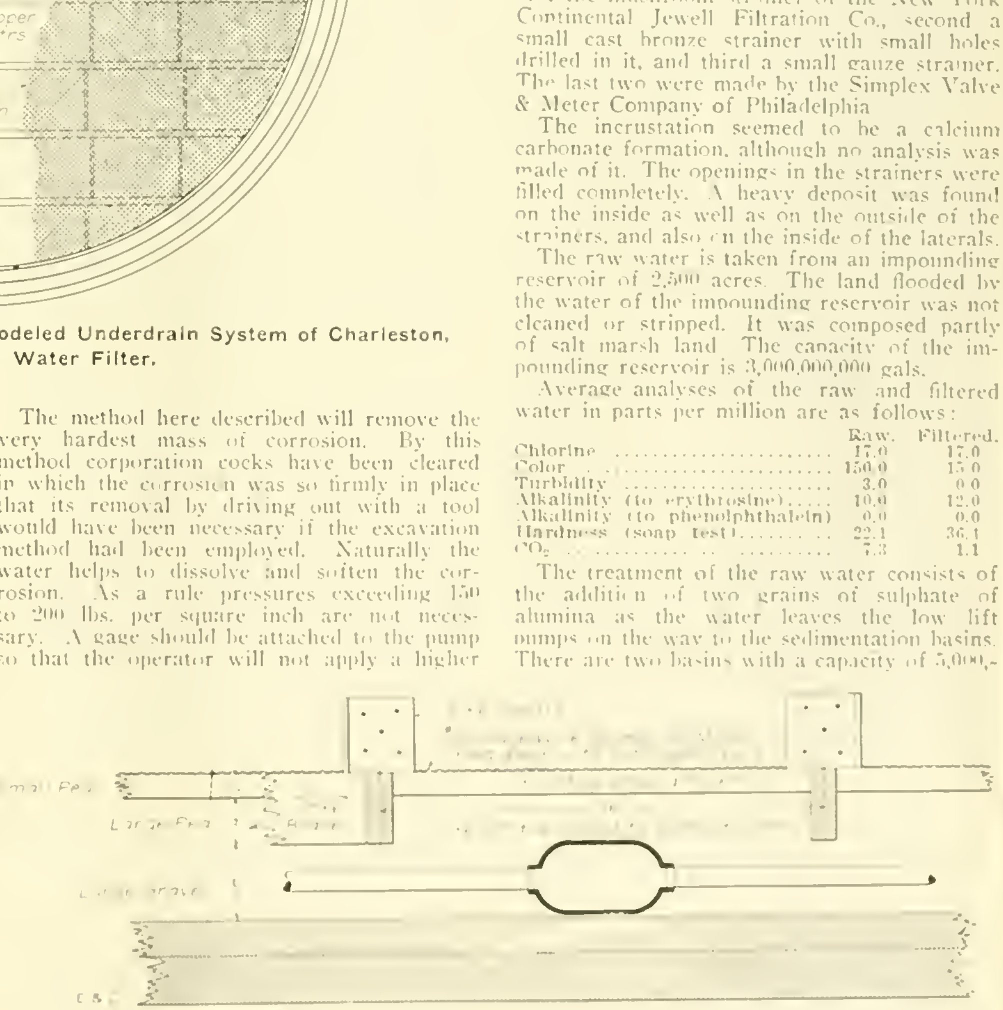


Fig. 2. Cross Section of Framing for Screen, Charleston Mechanical Filter.

ed lime are added. The lime solution is added to the filtered water in the flume which carries the filtered water to the clear water basin of 2,000,000 gals. capacity.

On finding the strainers incrusted two methods of cleaning were tried. One was to take the strainers out and clean by hand, the other was to immerse the strainers in a nitric acid solution. In 30 days the strainers were again incrusted. It came to a point of finding a substitute for the strainers or a new system of underdrains.

After much study, and consultation with the American Pipe and Construction Co. of Philadelphia, the following scheme was tried. It has proven a success.

All the lateral system was removed from the filters. New lateral pipes were capped and drilled with ¼ in. holes spaced 6 ins. center to center. These were placed in the filters with the holes turned downward. Then a system of stringers of wood was put in place and held by copper nails. In the bottom and around the laterals 6 ins. of ¾ to 1 in. gravel was placed. On this 4 ins. of large pea gravel, then 2 ins. of small pea gravel were placed. Over this was placed copper screening, 100 meshes to the square inch, nailed with copper nails passing through copper washers. The filters were filled to the required depth with sand. See Figs. 1 and 2 for full details of the new underdrain system. The new underdrain system here described has given entire satisfaction.

A Discussion of Engineers' Estimates as They Affect the Contractor.

Some of the deficiencies of the general run of engineers' estimates, from the viewpoint of the contractor on public works, were pointed out by Mr. P. R. Fletcher of Blue Island, Ill., in his presidential address before the Illinois Association of Municipal Contractors, at the 1914 annual convention held at Chicago on February 19. The following matter, of interest both to engineers and contractors, is taken from Mr. Fletcher's paper.

The United States Government, the legislatures of all the states, the officers of every city, village and town, annually levy an appropriation to pay the expenses of running their respective governments for the succeeding year. Every railroad company and industrial corporation in like manner makes an estimate and appropriation for their corporate expenses. This must be done in order to keep the cost of operation and maintenance in reasòn and to limit the entire cost to a fixed sum of money which they anticipate receiving.

In like manner the engineer in preparing his estimate of the cost of any improvement should with care and deliberation go into all details and arrive finally at a sum which will be sufficient to pay the total cost of this improvement and all expenses connected therewith. This estimate should be ample and liberal to allow for all the requirements of construction and the contingencies which might arise.

There is occasionally an estimate for public work in the territory in which I have operated that I would call reasonable; there are some that are fair, more that are low—and some, very low. Engineers' estimates are invariably low. I have, therefore, taken for my text the following clause, found in every special assessment proceeding in the state of Illinois: "I hereby certify that, in my opinion, the above estimate does not exceed the probable cost of said improvement and the lawful expenses attending the same and connected therewith."

In that one word "my" lies the authority for the estimate, the value of the estimate, and whether or not it is adequate for the construction of an improvement. By that one word the state of Illinois has given its authority to an individual to carry out its constructions and to prepare an estimate for vast sums of money. It has not hampered him except as to the word "exceed" and, as everything is left to his personal opinion, the value of that word is negative.

At the first sight of this certificate, one would think that this estimate must be exactly what the proposed work would cost, and could not exceed that sum. That is what it is intended to be, but, with the fluctuations of labor and material, and other causes, estimates vary a great deal, and if not made with due care, by an engineer who has had a great deal of actual experience and who has kept cost accounts on construction work, will often prove insufficient.

The city engineer who does all his work in one city becomes familiar with all the conditions entering into the cost of work in that city, and usually makes conservative and reasonable estimates. On the other hand, if he uses the same schedule of prices of labor and material upon going to some other city or village, the estimate may vary either way, and if too low may cause the non-letting of the contract. Again, there are many instances where the engineer has made fair and liberal estimates, according to his judgment, but has been compelled to lower them on account of the opinions of some members of boards of local improvement, who consider the estimate too high for that particular locality. Low estimates often result in the failure to let the contract and the abandonment of the proceedings, the letting of the contract above the estimate necessitating a supplemental assessment, or the letting of the contract to undesirable contractors.

Less than a decade ago estimates of cost, both by engineer and contractor, were generally mere guesses. Both, however, are beginning to realize the value of keeping construction costs. Both in estimating and in carrying on construction, methods and costs are taken into consideration, so that a better understanding now exists as to the difficulties to be overcome in construction work.

Though every engineer and contractor has a different method of making estimates of costs, in the main and important details they are similar. For example, any one estimating the cost of concrete per cubic yard can obtain prices from the dealers on the aggregate, sand, cement, steel and lumber. The exact amount of each material needed for a cubic yard can be calculated and set down without any guessing. There are only two items—the cost of mixing and placing and the cost of erecting and removing forms, both items of labor—that remain to be estimated. By consulting some practical concrete man who does this kind of work, or taking it from cost records, the estimator can ascertain the cost of these items. Adding to it allowances for plant and general expenses, the contractor's legitimate profit and discount of paper, he has a fairly accurate estimate. This method reduces guesses to a minimum, and instead of a guess in a lump sum, what little must be done is confined to a few items, generally a small per cent of the whole. Those who follow this method and do much estimating will soon learn the various man and machine units of work per hour or per day, so that the only uncertain element of labor cost will be the fluctuation of wages.

There are many conditions which affect the fluctuations of wages. Men may be plentiful when a job starts, but wages may increase, due to the beginning of other work in the same locality or generally good business conditions, and labor become difficult to obtain. In many cities, labor unions are in control of the situation and a union scale of prices is in effect and must be met. In many smaller villages and cities, no satisfactory floating supply of labor is to be obtained. The contractor must import laborers and provide a place to sleep and feed them, an added cost on small contracts. Wages paid to local labor are always higher for an outside contractor than the prevailing local scale, usually about a 25 per cent increase.

Local laws may also affect the labor costs. City ordinances imposing regulations on maximum loads, etc., may increase the cost of the work. The engineer must look fully into the wages paid in any locality where work is to be done and also into the many conditions which might arise.

IMPORTANCE OF ALLOWING FOR CONTINGENT COSTS.

In the case of materials to be used, there should never be any guess work as to their cost. Before the estimate is compiled, prices of all materials, based on the approximate amount needed, should be obtained in writing from the manufacturers or dealers. In the request for such prices, it should be stated what the materials are for, and about when and where they will be required.

Every contractor is familiar with the items of contingent cost encountered in nearly every specification. He reads them over and says to himself, "Oh, that isn't much; that doesn't amount to anything." It does not, if there were only one; but, taken in the aggregate and put into dollars and cents, it represents a large item of cost. If he does not figure it, his profit is reduced; and the probable reason he has never figured it out is that he could not get his bid under the engineer's estimate if he had—for the very reason that the engineer had omitted it also. Here is the big question: "Do engineers ever figure the contingent cost and add it to their estimated cost?" I believe that here is the true reason why many estimates are too low.

The following items of contingent cost enter into nearly every contract: 1, Expense of looking over the proposed work, attending the letting and closing the contract, if successful; 2, Attorney's opinion as to legality of the special assessment proceedings and bond issue; 3, Cost of the surety bond; 4, Cost of maintenance bond; 5, Cost of the liability insurance policy; 6, Moving outfit to and from the work; 7, Plant maintenance; 8, Tools lost, broken and stolen; 9, Wasted material; 10, Demurrage; 11, Losses due to weather conditions and damages of the elements; 12, Pay roll and expense of outfit in rainy weather; 13, Pay roll of superintendent and foreman for three months of the year, when not at work; 14, Local charges for use of water; 15, Broken drain or water pipes; 16, Unforeseen damages to walks and private property; 17, Cost of inspection of materials; 18, Stakes and help for the engineer; 19, Barriers and red lights; 20, Lost, torn or damaged cement bags; 21, Being a good fellow.

I could explain each of these items more fully, but the mere mention of them will suggest many instances to every contractor. Every one of these items enters into the cost of an improvement. Each is as much an item of cost as labor and material. Taken separately, they amount to but a small sum, but in the aggregate they total from 10 to 15 per cent of the cost of an improvement. How many engineers take cognizance of these items and include them in their estimate? Yet their specifications and contracts call for them.

No business can succeed without profits, and no contracting company can succeed, and stay in business, without them. This leads to the question, "What constitutes a fair net return on municipal work?'

The contractor's profit depends on the price he bids for the contract, and often varies for the same class of work. In early spring he may bid very low, to get his organization ready for the season, especially if he carried no work over from the preceding year. Later on he, perhaps, obtains contracts at legitimate prices, thus averaging up on the year's business.

On force account work, with every facility at hand to do work, the usual allowance for use of tools and profit is 15 per cent. Gillette figures profit on the basis of 10 per cent on the cost of material and 33½ per cent on the cost of labor. This would probably average about 20 per cent on the whole contract.

Contractors for municipal work are paid in cash or bonds. The occasions when they are paid in actual currency are so rare that they need not be discussed. These bonds are of two kinds: the municipal bond, guaranteed by the city or village as a whole; and the special assessment bond, issued by the city or village in anticipation of the collection of the tax upon each lot or parcel of land benefited by the proposed improvement. Here are two classes of bonds issued by the same city or village for the payment of the same class of work, each drawing 5 per cent interest, each taken by the contractor at par, yet of very different market value.

The special assessment law of the state of Illinois is considered very much superior to the similar laws in Iowa, Wisconsin and Indiana, irrespective of the fact that where only 5 per cent interest is paid in Illinois, 6 per cent is paid in these other states. The special assessment bonds of these states readily sell for par, owing to the increased rate of return on the investment, and the public buys them for the reason of that particular attraction. The Illinois bonds assessed on relatively the same kinds of property are sold at a discount of from 5 to 10 cts. on the dollar, in order to net an average of 6 per cent or more for the time that they have to run.

Here, then, we see the property-holders paying nominally 5 per cent on an inflated cost of an improvement according to the state law, but paying 6 per cent or more on the actual cost. This difference of cost of the work, ordinarily termed discount, must be added to the engineer's estimate as a part of the cost of an improvement. This discount is not always 5 per cent, but varies as the money market changes. When money is plentiful, bonds can be marketed at 95, but during the last year many have sold for less, if they could be sold at all.

Special assessments are levied against every lot or parcel of land benefited by the improvement, according to such benefits, and the assessment must be equitable. The amounts so assessed are spread irrespective of the value of the lot or the owner thereof. If the property assessed is improved, the value is greater and the security behind the bond is increased. If the property is vacant and poor in value, it has a tendency to decrease the security of the bond, unless it is so located that its improvement or the growth of adjoining property enhances its value.

Again, many assessments are spread against property which has been sold for taxes and been forfeited to the state. In this case the state pays no special assessments. Hence, in looking over proposed improvements before the letting, the contractor usually examines the assessment and if he finds property forfeited to the state it reduces the amount available for the construction of the improvement by the total amount of the forfeitures. If the estimate, after taking off the forfeitures, is too low to do the work, the contractor either must not bid at all or else must bid above the estimate. From these facts it follows that it should be the duty of the engineer to look over property before making his estimate, to ascertain its condition, and make his estimates accordingly.

THE VALUE OF BONDS.

The first duty of the contractor, when he receives bonds for work, is to see that they are good. He must depend upon the expert attorney to advise him. But such advice, however favorable, will not force a property holder to pay his tax for an improvement, nor will it force the state to pay a tax upon a lot which has been forfeited to the state for nonpayment of taxes. No special assessment bond becomes forfeited if every lot or parcel of land pays its tax. If property that is assessed for special assessment improvements is clear as to title and is worth as much or more than the assessment, there is not much doubt of the assessment being paid. It is this doubt or possible chance of a forfeiture that bankers and bond-buyers shy away from. They use it as a club to force the contractor to sell his bonds at their price. After having the opinion of the expert and of their own legal talent as well, they send representatives to look over the property assessed, and if there is no possible chance of a loss, the contractor may be offered 95 per cent, if 30 per cent of the property is vacant, 92 per cent. If there is a forfeited lot, he will get no offer at all.

I am going to suggest two remedies that I am positive, if made law, would have a good effect on the market value of these bonds. As they depend upon the engineer's estimate they may properly be included here.

First, the engineer's estimate should be ample; enough so that after paying the contract and the 6 per cent allowed by law for the expenses of making the assessment, court costs, etc., there would be left a residue. Our present law requires that after the cost of an improvement is determined, the residue shall be rebated to the property holders before they pay their assessments. They pay only what is actually required. My suggestion is that no rebate shall be made until the final bond is paid. I contend that if the property owner elects to take five or ten years to pay his debt, he should pay all expenses connected with it. I contend that the contractor should not assume the collection nor stand the responsibility of the non-collection of special assessments. If there were a residue or reserve left in the special assessment fund, over and above the obligation or requirements of that assessment, it would undoubtedly go far as a guarantee toward insuring the full payment of every bond and the interest thereon, in case of a possible default in the payment of some installment.

Secondly, 6 per cent is allowed by law to reimburse general fund accounts for cash advanced on account of the expenses of levying all assessments, and this 6 per cent is always drawn from the first installment. Thereafter there is no fund to pay any expenses which may develop on account of the special assessment. If some of the property holders pay their assessment in full, the interest stops on their portion but the bonds continue to run until called. Here is a loss of interest and no fund to pay it. There might be a default on some payment of an assessment and no fund to pay it. The bonds are obligations of all the property holders, and inasmuch as the state has seen fit to assess them 6 per cent for the making of an assessment, I can see no valid reason why another assessment of, say, 5 per cent. should not be levied and held as a sinking fund or reserve to pay the loss of interest and forfeiture, if any, and at the final payment of the last bond to be rebated to the property holders.

The assessing and creation of this reserve will, in my opinion, bring the value of special assessment bonds to a higher plane as marketable and investment securities. Contractors who, in bidding heretofore have figured a return of 90 to 95 cts. on the dollar, would, if these provisions were a law, be able to figure on the basis of nearly par. If the creation of this reserve acts as I think it will, I cannot see that the property holder eventually pays any more than he does at the present time. What he now pays for as discount will be offset by the increased value of the bond.

The present state law requires that interest shall be paid upon all of an assessment except improvements put in upon the one-year plan. There are a great many such improvements, so small in total amount as to dollars and cents that it is impossible to make them in 5 or 10 installments, where each installment must be a multiple of $100. Very often these improvements are put in in the early part of the season or late in the fall, and in either case the contractor must wait a year or year and a half for his money without interest. I believe the laws should be amended so that this kind of paper would bear interest, putting it in the marketable class of securities.

There seems to be a dark cloud in our special assessment law in regard to interest on the first installment where there are deferred installments. The ordinance usually sets forth that all installments except the first shall draw interest at the rate of 5 per cent, yet all municipalities collect the interest on the first as well as on successive installments.

Some municipalities issue vouchers on the first installment which bear no interest, but more of them issue those bearing interest. The former apparently have the law with them; the latter do it because they think it fair and honest, believing that as long as they collect the interest on the installment they should in turn pay it out on the voucher or bond. The special assessment law should be so amended that all installments shall bear interest and that all vouchers issued against the first or succeeding installments shall draw interest.

Summing up, an engineer's estimate, to be fair, conservative and reasonable, must be based on the following:

1. The actual cost of material and labor.
2. Contingent costs, amounting to from 10 to 15 per cent of the above.
3. Contractor's profit, amounting to from 10 to 30 per cent of the first two items. This will vary according to whether the contract is hazardous and difficult, requiring a large amount of special equipment, or whether it is comparatively easy, requiring but little equipment.
4. Allowance for converting the bonds into cash, amounting to at least 10 per cent of the total amount of the contract.

Drainage and Irrigation

Synopsis of Hearings on the Present Status of Reclamation Projects.

The subcommittee of the committee on appropriations in charge of permanent appropriations has presented a majority and a minority report on its findings from the evidence developed by hearings. The subjects discussed in this report are of direct interest to irrigation engineers, and we reprint the report in full as follows:

MAJORITY REPORT

The hearings thus far have shown the need for closer supervision by Congress over the expenditures of the Reclamation Service. The reclamation act has been in existence 12 years. Irrigation projects have reached a state which is described in current engineering journals as "stagnation."

Thirty-one projects, including three Indian projects, have been begun. Only two or three are nominally completed. The rest are in various stages of construction, some are less than 10 per cent completed.

The gross area included in the projects is 3,000,000 acres, but by the elimination of non-irrigable land the net area is 2,117,966 acres. Of this area about 700,000 acres are now under cultivation and the Reclamation Service is prepared to furnish water to about 500,000 acres more for which at present there is no demand by settlers.

The original reclamation act, passed in 1902, set aside the receipts from the sales of public lands in the States of Arizona, California, Colorado, Idaho, Kansas, Montana, Nebraska, New Mexico, North Dakota, Oklahoma, Oregon, South Dakota, Utah, Washington, and Wyoming as a reclamation fund. In the debates in Congress at the time it was estimated that this fund would amount in the 10 years following the passage of the law, to about $30,000,000. It was the plan of the statute that the cost of each project should be repaid to the fund by the settlers in 10 annual installments, and the fund thus renewed should be used in the construction of other projects. The whole liability, actual and contingent, on the Federal treasury was limited by plain intent to this fund. While the Reclamation Service, with the assent of the Secretary of the Interior, was clothed with power to engage in the construction of irrigation projects without limit on the number or cost, the plain duty was imposed not to start so many projects or increase their cost to such an extent that they could not be completed with the funds set apart by Congress. The intent of the law has been violated in every important respect. The estimates have not been kept within the funds set apart, and the revolving feature of the fund has been abandoned.

The original estimate of receipts from the sale of public lands for the years 1901 to 1910, inclusive, was $30,000,000. The actual receipts were more than double that sum, $65,370,802.75. From 1901 to 1913, inclusive, the receipts from public lands turned into the reclamation fund were $80,901,759.94. This was greater than could have been expected, but was not great enough to keep pace with the increased cost of the irrigation projects. The capital investment in reclamation projects to the present fiscal year is $110,000,000. It will cost, at the lowest estimate, $40,000,000 additional to complete the incomplete projects. Thus the lowest estimate of the cost of completing the present incompleted projects is five times as great as the fund originally contemplated. But the estimate of $150,000,000 to complete the present projects is too low. It will approach more nearly $200,000,000.

The act of June 25, 1910, provided that a board of army engineers should examine the projects and report which of them were worthy of completion and the estimated cost to complete. The report of the army engineers was sent to Congress January 6, 1911, and is House Document No. 1262, Sixty-first Congress, third session. At the date of that report the capital investment in the projects was only $53,781,302.88. The total cost of completing the projects according to the army engineers was placed at $150,277,090. It is apparent that the projects can not now be completed for that sum. In the four projects which have been examined in detail the final estimate now made by the reclamation engineers exceeds in every case the estimate of the army engineers. The following table gives the original authorized expenditure, the estimated costs set by the army engineers in 1910, and the estimate now fixed by the reclamation engineers in 1914:

	Authorized cost.	Army engineers.	Final estimate.
1. Satt River..	$3,806,000	$9,665,000	$11,862,972.46
2. Yuma	3,000,000	7,700,462	9,899,495.96
3. Orland	650,000	607,335	936,000.00
4. Gnd. Valley	3,600,000	3,621,663	4,595,020.00
	$11,056,000	$21,591,460	$27,293,488.42

It will thus be seen that the final estimate now given by the reclamation engineers is two and one-half times the authorized cost and is nearly $6,000,000 greater than the estimate of the army engineers in 1910. It is safe to say, therefore, that the projects will cost more nearly $200,000,000 than the $150,000,000 estimated by the army engineers. The reasons for this increase are various. One element is the increased cost of construction, labor, materials, etc. This is not the principal element, however. The largest factor is that the government has undertaken the construction of distributing ditches, laterals, power development, pumping and other supplemental features which were, according to the original plans, to be assumed and constructed by the landowners themselves. Experience has developed an additional problem that must be handled by the Federal government. Most of the projects have developed seepage by which the land becomes water-logged and swampy. Large bodies of land are thus made unfit for cultivation, and, of course, unable to contribute to the cost of the project until restored by a system of drainage. This drainage may be done in some cases by main ditches, but in some cases must go to the extent of tiling each farm unit affected. In all it represents a substantial increase of the cost of the project

The evidence indicates that the greater percentage of the land under the irrigation projects is still in the hands of speculators, only the smaller fraction being in the possession of actual settlers.

By the act of June 25, 1910, Congress authorized the advance of $20,000,000 out of the treasury to the reclamation fund, to be paid by certificates of indebtedness. The advance has been made to the full amount, but no certificates of indebtedness or bonds have been issued. The last of this money will be spent the current fiscal year. The receipts from public lands are steadily going down. They reached their highest point in 1908, when about $10,000,000 was turned into the reclamation fund from that source. In 1914 the amount had fallen to about $4,000,000 and will decrease still further. The receipts from the sale of public lands, together with repayments by settlers into the reclamation fund, will not exceed $7,000,000 a year in the immediate future. This will not be sufficient to complete all the projects now undertaken at the present rate of progress. At the same time constant local pressure is being exerted to induce the government to begin new projects or to take over bankrupt private projects or Carey Act projects.

Even so recently as September 23, 1912, Secretary Fisher authorized the construction of a new project—Grand Valley in Colorado—which will cost $4,500,000 and which is now proceeding as rapidly as the others. The Secretary should have known that two years before—in 1910—the Board of Army Engineers had reported that the service was $40,000,000 short of the amount of funds necessary to complete the projects then undertaken.

In view of these facts the committee believes that no new legislation should be enacted relating to the Reclamation Service without providing:

First. That annual estimates be submitted to Congress of the expenditures of the Reclamation Service; and

Second. That no new projects be begun without express authority of Congress.

Committee: Wm. P. Borland, George White, Wooda N. Carr, B. F. Mahan.

MINORITY REPORT.

The undersigned members of the subcommittee of the Committee on Appropriations which has been conducting hearings with a view of securing information relative to projects constructed under the reclamation act can not agree with many of the statements contained in the so-called "Synopsis of Hearings" signed by the chairman and majority of the subcommittee or approve the conclusions drawn from such statement. If the majority were accurate in their statement of facts their recommendations would afford an altogether inadequate remedy.

The hearings on which the so-called "synopsis" was based were neither detailed nor exhaustive. They did not cover any one project of the 31 on which work has been done thoroughly or completely. A majority of the projects with regard to which inquiries were made were treated only in a general way and very briefly, and a number of projects were not even referred to. It is perhaps not strange that such limited investigation, unaided by familiarity with the subject, should have led the majority into errors of fact and opinion.

The synopsis contains a number of statements of alleged facts which would present a serious situation if justified by the hearings, the more important of which, taking them up in logical order, are as follows:

1. That the intent of the reclamation law has been violated in every important respect and especially in undertaking too many projects.

2. That "the estimates have not been kept within the funds set apart."

3. That "the revolving feature of the fund has been abandoned."

4. That "the greater percentage of the land under the irrigation projects is still in the hands of the speculators, only the smaller fraction being in the possession of actual settlers."

5. That "irrigation projects have reached a state which is described in current engineering journals as stagnation."

6. That the Reclamation Service is prepared to furnish water to about half a million acres "for which at present there is no demand by settlers."

The conclusion of the majority is as follows:

In view of these facts the committee believes that no new legislation should be enacted relating to the Reclamation Service without providing, first, that annual estimates be submitted to Congress of the expenditures of the Reclamation Service, and, second, that no new projects be begun without express authority of Congress.

Taking up these matters in the order stated, the first allegation is that the law was violated by taking up too many projects. The reclamation law did not limit the discretion of the Secretary of the Interior in the number of projects to be undertaken, but unquestionably did intend to limit the liability of the government to the receipts from the sale of public lands, whatever that might be. Public lands receipts increased rapidly after the passage of the reclamation law and from time to time new projects were undertaken. The receipts from this source turned into the reclamation fund have amounted, as the majority state, to nearly $81,000,000.

When these projects were undertaken the majority of them were planned on a much smaller scale than they were afterwards carried out or are proposed to be extended. Nothing could be more in harmony with the letter or the spirit of the law than the planning, and, as funds were available, the extension of projects to their widest possible extent. The distribution of the projects throughout the arid region in the states named in the act was clearly the intent of the law. By a later enactment Congress brought Texas within the law, and thus laid an additional burden of about $10,000,000 on the reclamation fund, with no increase of revenue.

Because settlers had gone upon lands withdrawn under the reclamation act in anticipation of their early reclamation Congress made a loan of $20,000,000 to the reclamation fund. This was needed not because there were too many projects, but because under the plan which the law originally provided settlers were justified in expecting speedy reclamation of their lands.

With the fund available and the returns provided by law, it was well within the discretion of the Secretary of the Interior to start a number of projects, thus providing opportunity for each to gradually develop in accordance with the surrounding conditions. Experience has shown that it would have been unwise had the Secretary built and completed, say, only one-third of the projects which have now been successfully initiated. Neither the law nor the spirit of the law has been violated, as in each case funds have been available for completion of the first and important units of the work.

The statement that it will cost at the lowest estimate $40,000,000 additional to complete the uncompleted projects is based on the assumption that all of the new units proposed or suggested will be ultimately built. The evidence has shown that, while most of the proposed units are desirable, they may not be built by the Secretary of the Interior unless certain favorable conditions continue or develop.

The units of each project already built are in many cases more or less independent of proposed extensions included in this $40,000,000 or any other sum. The conditions are similar to that of a railroad system, where the government has built a main line and some of the feeders. While ultimately other feeders are desirable and should be built, the fact that they are not immediately constructed does not prevent the main line and other portions already in use from being of great benefit.

It is suggestel that the estimate of $150,000,000 to complete present projects is too little, and that it will approach nearer $200,000,000. There is nothing brought out in the hearing to indicate this likely to be the fact. Each of these projects is a growing, developing enterprise, which is being added to from time to time by the Secretary of the Interior as needed, and which it is to be assumed is being limited by the Secretary of the Interior in accordance with careful studies of prevailing conditions.

The estimate of $7,000,000 per year from the sale of public lands in the immediate future, while not sufficient to complete all of the units which have been proposed at any early date, yet will enable progress to continue and to a considerable extent will keep up the growth of communities.

It is true that constant local pressure is being exerted to induce the government to begin new projects or to take over bankrupt private projects or Carey Act projects. This pressure will doubtless continue It is the strongest possible tribute to the fact that the work undertaken under the terms of the reclamation act is appreciated, that it has been successfully conducted, that it is steadily building up communities, and that the use of the reclamation fund is adding to the number of homes available for citizens of limited means.

The second charge is that "the estimates have not been kept within the funds set apart." We are not entirely clear as to what the majority mean by this statement, but assume their contention to be that some of the works have cost more than they were estimated to cost. This is true, particularly with regard to some of the earlier projects, owing to a general increase in construction cost throughout the western country soon after the reclamation law passed, and also to the fact that the works put in were of more substantial and permanent character and better finished than private works on which the estimates were in some cases partly based.

The principal increase in the cost of projects above the original estimate, however, is due not to the increase of unit cost, but to the increase of units, enlargements, and extensions above the original plan, so as to utilize all available water supply and irrigate the greatest possible acreage. This is certainly in accordance with the spirit of the reclamation law.

In the table giving the original authorized expenditures it should be remembered that in many cases the amount originally set aside was for a smaller and less extensive work than that subsequently authorized by the Secretary of the Interior after full consideration of reports made by his engineers and conference with citizens and entrymen. Taking the Salt River project for example, the preliminary estimate was for a relatively small reservoir; the estimate prepared by the army engineers was for a larger reservoir and distribution system. In years which have elapsed since the army board report was prepared the Secretary of the Interior has concluded to build a more complete distribution system and take up other work which seems wise. It should be noted that of the increase of about $3,000,000 between the estimate of the army engineers and that now in hand, $1,000,000 has been paid by the people on the project for power plant.

The third indictment is that "the revolving feature of the fund has been abandoned." On what statements made at the hearing or information from any source the majority base this extraordinary statement we are at a loss to know or even guess. On every one of the projects where water is being furnished payments are being made, and up to March 1 last $3,183,658.50 had been paid into the reclamation fund by settlers on account of construction and maintenance charges. No one is proposing, no one is suggesting, so far as we know, the suspension of payments or the modification or abandonment of the revolving feature of the act. While the testimony before our subcommittee was to the effect that the settlers were making good progress and the projects were coming under cultivation rapidly, the officers of the Reclamation Service called attention to the difficulties to be met and overcome, the great expense to the settler of making improvements and of getting his lands in condition to grow crops, and therefore the need of an extension of the period of payments as proposed in a bill now before the House of Representatives. Such legislation would, in our opinion, be favorable to the revolving feature of the fund by granting the settler needed time in which to meet his obligations.

The fourth charge, with regard to condition of reclamation projects, is that "the greater percentage of the land under the irrigation projects is still in the hands of speculators, only the smaller fraction being in the possession of actual settlers." We find it difficult to understand where the majority could have obtained the misinformation on which to base such an allegation. There was certainly nothing brought out in the hearings to even suggest such a statement unless the majority designate as a speculator every man who, holding an irrigated farm, might be willing to sell if he received a good price.

On several of the great projects, such as the Shoshone in Wyoming, the Minidoka project in Idaho, and others, all or most of the lands were public lands taken up by homesteaders under the reclamation law. Are these the men the majority refer to as speculators? It's true that on some of the projects, such as the Salt River project, there were considerable areas which, when the project was undertaken, were in private ownership in large bodies. These have been divided up and sold in holdings of from 40 to 80 acres. As no one owner can receive water for more than 160 acres, these larger holdings must necessarily be broken up. The bill for the extension of payments, which has been heretofore referred, contains several provisions which would make subdivision of lands into small units even more speedy than at present. While the hearings before our subcommittee did not go into this matter in great detail, our information is that probably less than 5 per cent of the land under the irrigation projects is in the hands of men who are trying to speculate in this land. Over 95 per cent of the area is unquestionably in the hands of men who are complying, or have complied with the conditions requiring settlement or residence in the vicinity.

For example, as above stated, practically all of the holdings which may be called speculative in the Salt River Valley have been eliminated. Those at Yuma and Orland are being disposed of. In Colorado under the Uncompahgre and Grand Valley projects the land is almost wholly in the hands of small owners. The same is true in Idaho, where the only notable large holdings are those of the state which under the extension bill would be divided up. In Montana also the farm units are usually from 40 to 80 acres with an occasional 160-acre tract. In Wyoming all of the lands are in the hands of homestead settlers in tracts of from 40 to 80 acres. In Nebraska the units do not average over 80 acres. In Nevada less than this. In New Mexico and Texas there are a few large holdings, which, of course, can not obtain water until subdivided. In South Dakota the irrigable lands are in 40 or 80 acre tracts, and in Washington the average size is still smaller. The average size of farms in all government projects is about 50 acres, of which about 30 acres have been irrigated.

A further statement is that irrigation projects have reached a state "described in current engineering journals as stagnation" It's true that on some projects work of extension is not progressing as rapidly as the people in the vicinity would like to have it progress. In the matter of improvement and advance by settlers we have heard nothing to indicate that stagnation exists, although many settlers are finding it difficult to meet their payments The records show that the cultivated area on each project is increasing yearly at a goodly rate; that settlers are steadily and progressively improving their farms; and that important communities are being built up on reclamation projects. To say that stagnation exists on these projects is to confess lamentable ignorance in regard to them

Another statement of the majority is "that the Reclamation Service is ready to furnish water to about 500,000 acres, for which there is no present demand by settlers" This statement that settlers are declining to take lands for which water has been furnished is curiously at variance with the other statement of the majority that most of the land is in the hands of speculators. The majority of the subcommittee has evidently, through lack of information on the subject, fallen into the error of confusing cultivated land with entered or appropriated land. It is true that of approximately 1,200,000 acres furnished with water but 700,000 acres are under cultivation. The major portion of the remainder is, however, also in farms, and represents the portion of the farm not yet brought under tillage. Out of more than 20,000 farm units which were available for homestead entry and are furnished with water, less than 420 remain untaken, and the probability is that most of these will be entered and settled upon before the close of the present season, as nearly half of this small remainder have been subject to entry less than a year. It is true that not all of the land entered is under cultivation, for it takes the settler a number of years, after he is furnished with water, to clear, level, and plow up his land and get it under cultivation.

As we can not agree with the accuracy of the statements on which the majority base their recommendations, we can not approve their recommendations. Under the reclamation act a detailed statement of annual expenditures is now furnished in the annual reports, and may be considered by Congress at any moment, as they have been in the past, by the Irrigation Committees of the House and the Senate. There can be, at least there should be, no objection on the part of anyone to have the Reclamation Service make more complete and detailed reports to Congress than it now does, if that should be deemed necessary. We doubt, however, if anything would be gained by submitting estimates for the work to be undertaken and await appropriations by Congress. As to the second recommendation, that no new project should be undertaken without express authority of Congress, that would seem to be unnecessary. No new project should be or is likely to be undertaken until present projects are well toward completion. No new project has been taken up since the report of the Army board. The Grand River project in Colorado, sometimes referred to as a new project, was in fact treated by the Army board in 1910 as a project under way.

The history of river and harbor appropriations does not furnish arguments in favor of a like system in regard to reclamation projects. If the selection of projects were to be made by Congress more rather than fewer projects would be authorized. We would recommend, first, the passage of the bill providing for the extension of the period of payments on reclamation projects and for other needed changes in the reclamation law and believe it would be wise to have some committee of the House authorized, if that should be deemed necessary, to conduct annual hearings relative to the operation of the Reclamation Service. Such hearings would tend to familiarize Congress with the operations of the service, with the conditions of the projects, and would, we believe, be helpful and useful. Committee: F. W Mondell, C. R. Davis, W. S. Vare

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Large Pumping Plant for Egypt.—The Egyptian Government has just awarded a contract to Sulzer Bros., of Winterthur Switzerland, for a new drainage pumping plant to be erected at Baltim, near Cairo. The cost of machinery, not including buildings, masonry and canals, is to be about $900,000. Besides a lot of electric lighting and auxiliary machinery, there are to be eight 400-HP. Diesel crude oil engines; six 175-HP. steam engines direct connected to centrifugal pumps and four water-tube steam boilers, having a total heating surface of 5,700 sq ft.

The requirements are to pump 7,100,000 cu. ms. in 24 hours against a static head varying from 2.1 to 3.9 meters. This is equal to about 2,000,000,000 gals against a head of 6.9 to 12.8 ft., and theoretically figures 4,200 static HP. The combined rating of the engines is 6,050 HP., with considerable overload capacity. The Government supervising engineer is Sir Arthur Webb.

Method of Handling Subterranean Water in Constructing a Core Wall for an Earth Dam.

The Hinckston Run dam, built 13 years ago near Johnstown, Pa., was one of the first of American dams at which grout injection was employed to close underlying rock strata against seepage under the dam. Until recently no account of this early use of grout injection has been given out, but the methods and results are of such interest that, despite the age of the work, they are summarized here from a letter by Mr. Harrison Souder in Proceedings American Society of Civil Engineers, Vol. XL, p. 1569.

The orginal Hinckston Run project called for an earth dam, 60 ft. high, to retain some 400,000,000 gals. of water, with a depth of 45 ft. at the breast. The intention was to build a dam with a clay core, but, as an unlimited quantity of cinder from the steel plant was available, it was decided, after the work was started, to use this as backing for the dam, in place of earth, and eventually to fill the whole valley below with this material, thus rendering the structure practically unbreakable. In view of this and the additional expense incurred in making the cut-off tight, the proposed height of the dam was increased to 80 ft., and later to 85 ft., above the original creek level. This gave a total maximum height above the bottom of the core-wall ditch of 112.8 ft., a depth of water at the breast of 73½ ft., and a capacity of 1,100,000,000 gals. The lake thus formed is 1¼ miles long. The water-shed above the dam is 10.75 sq. miles.

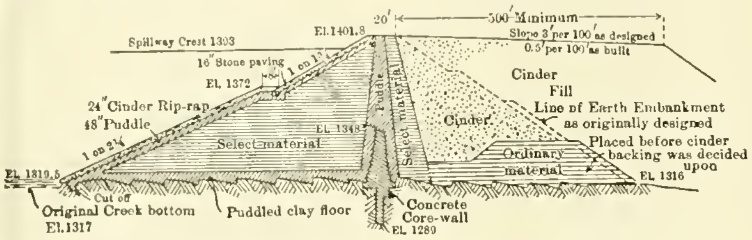


Fig. 1. Maximum Cross Section of Hinckston Run Dam.

The cross-section of the dam as built is shown by Fig. 1. The lower inner slope is 1 on 2¼, with 4 ft. of puddle and 24 ins. of cinder riprap. The slope above the berm is 1 on 1¾ with puddle lining diminishing to 2 ft. thick at the top. The facing is hand-laid stone paving. The puddle wall is 16 ft. thick at the top of the concrete core-wall, and diminishes to 4 ft. at the top of the dam.

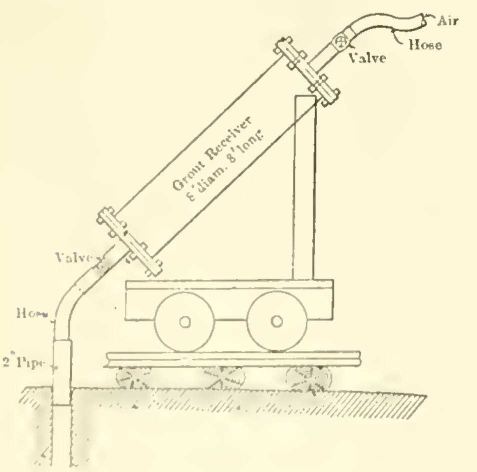


Fig. 2. Grouting Device Tried and Abandoned

When the core-wall ditch had been carried down to hard rock at what was thought to be the proper depth, some test holes were bored through the bottom to determine the character of the rock below. This disclosed a layer of hard sandstone a few feet down, with considerable water flowing below and above it. It was decided to deepen the ditch considerably, in order to get below any rock strata that might come to the surface within the flooded area, and to substitute a concrete core-wall for the clay one originally proposed.

An air compressor plant was installed. This was a 14x18-in. Ingersoll-Sergeant machine capable of driving two rock drills and six pneumatic rammers. These rammers were used in tamping concrete and also in puddling clay in such places as could not be covered by a 10-ton steam roller which was also supplied at this time in place of the 3-ton horse roller in use.

The finished ditch averaged in depth 25 ft. below the grubbed surface in the valley, but reached 50 ft. at the ends. The shale was excavated with picks, but the harder rock was loosened with light charges of dynamite, care being taken not to shatter the foundation or open up the seams. The advisability of cutting off the underflow to as great a depth as possible was realized, and it was determined to remove the shale down to the sandstone and try to cut off the flow below by forcing in cement grout under air pressure.

GROUT INJECTION.

Drill holes, 2 ins. in diameter and from 10 to 16 ft. deep, were drilled through the rock, averaging about one hole per linear foot across the valley. Iron pipes, 2 ins. in diameter, 18 ins. long, and threaded on one end, were cemented into these holes. Portland cement grout was poured into them and then air at a pressure of from 30 to 60 lbs. was applied. The first holes were approximately 6 ft. apart. They were drilled generally 10 ft. below bed-rock.

Figure 2 is a sketch of the first contrivance or receiver devised for applying the grout. It was a cylinder, 8 ins. in diameter and 6 ft. long. A screw flange was provided at top and bottom, and a steel head-plate was bolted to each end, with rubber gasket packing. The top bolt holes were open to allow quick removal of the lid. A 2-in. pipe with plug cocks was provided at the top and bottom. With a short hose, the cylinder was coupled to the pipes in the holes. The cylinder was filled with grout; the valve was opened; the grout ran into the drill holes, and air pressure was then applied at the top. The contrivance was mounted on a truck running on a track in the bottom of the ditch. After trial it proved to be too slow and cumbersome, and another method was devised and operated satisfactorily. Figure 3 is a sketch of this final arrangement. The method of grouting was as follows: A 1-in. pipe, long enough to reach to the bottom of the hole, was inserted and air was applied to blow out water and dirt. Then a tee and the pipe, *C*, were attached to the tube in the drill hole with a sleeve union, as shown. The cock, *A*, was closed, the cock, *B*, was opened, and air was applied, thus forcing the water out of the hole and into the crevices and near-by drill holes; meanwhile, the pipe, *C*, was filled with grout, and the air hose was connected at the top. Then *B* was closed, *A* was opened, and air was gradually applied at the top of *C*, forcing the grout down into the crevices. The pipe was refilled about every 10 minutes until the hole would take no more. The apparatus was then removed and a cap was screwed on the tube in the grouted hole. After a given length of ditch was grouted in this way, and sufficient time had elapsed to allow the grout to set, test holes were bored within the grouted area and the process was continued until there was no indication of water flow below the bottom. The greater part of the bottom was grouted successfully in this way, but, as explained later, the grouting scheme was abandoned where the core-wall ditch entered the side-hills.

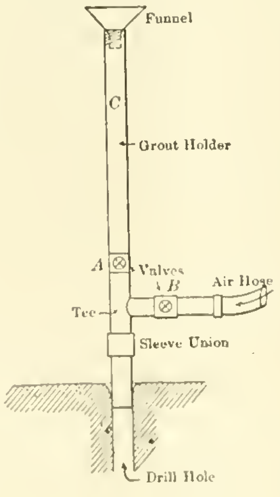


Fig. 3. Grouting Device Employed.

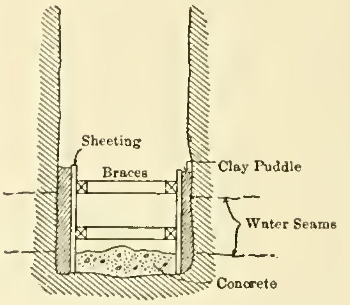


Fig. 4. First Method of Handling Water in Cove Wall Ditch.

HANDLING FLOW FROM SEAMS.

At the east end three large vertical open crevices were uncovered. The flow from these was so considerable that it was necessary to put in a force pump with a 4-in. suction and a 3-in. discharge, in order to keep the ditch clear. As the grouting method, after continued trials, did not give satisfaction at this end, it was decided to take up the bottom until the principal water strata were reached. This was done, and concrete was placed in the ditch, all the walls being plastered first with a rich cement mortar worked in with trowels. Two coats of plaster were applied on the north or reservoir side and one on the south side. The suction pipe of the pump was built in concrete, and carried up with the wall. The strong flow of water in this section of the ditch made it difficult to place the concrete for the core-wall with-

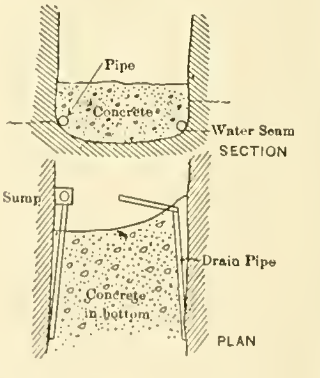


Fig. 5. Second Method of Handling Water in Cove Wall Ditch.

out having the cement washed out before it set.

At first, the method indicated in the sketch, Fig. 4, was tried, namely, a line of 1-in. sheeting was placed as shown, and clay was rammed between the sheeting and the rock to stop the water flow. Concrete was placed between the sheeting which was raised gradually as the concrete was carried up. This did not prove wholly satisfactory, and the method of piping the water directly to the sump was adopted, as shown in Fig. 5. This proved successful. Where there was too great a flow of water, plastering could not be done, but considerable neat cement was dumped in along the walls with the concrete and well worked in. After the concrete was well set, the pipes were plugged and the flow of water was shut off. As the concrete was carried up, the water came out of the ditch walls higher up, showing that the underflow had been intercepted. The core-wall ditch was extended well into the hills on each side of the valley, and test holes were bored for water, none being found, because, after a certain distance, the rock became hard and massive and free from seams. At the west end of the ditch, there was as much trouble with water. It would seem that the underflow of the whole valley was concentrated at this point, the rest of the ditch having been grouted and the flow cut off.

The drill-hole grouting method failed in the west side of the valley, as in the east, and here the bottom was also taken out, down to the water strata, and the water fought inch by inch by piping it from the streams to the sump, as before described, and the ditch was completely filled with concrete.

The proportion for the concrete in the core-wall was 1:2:5, generally; but, at the bottom, it was much richer in cement, which was not spared in efforts to make a tight job. Near the crevices a proportion of 1:1:2½ was used. These proportions had to be varied, also, to suit the sizes of the stone supplied, which varied from ½ in. at times to 3 in. The top section of the wall was made of 1:3:6 and 1:3:7 cinder concrete. The concrete was mixed in a machine of the continuous-mixer type, consisting of a long square revolving box with a helix at the back. The machine was not wholly satisfactory, but was the best to be had. The concrete was received in ½-cu. yd. dump buckets, carried on small trucks running on light track to the derricks; it was then lowered to the bottom of the ditch and dumped, as it was not thought advisable to drop it from any height. At first a middling dry concrete was required, but, later, a wet concrete was found to give the best results and was finally adopted for the remainder of the work.

After the core-wall reached above the surface the faces received two coats of plaster, one inside against the wood forms and another after the forms were removed. For the upper part of the wall, the plaster coat on the down-stream side was omitted; and, on the up-stream side, a cement wash applied with a brush was substituted for the second coat of plaster. Where sections of the wall joined, bonding grooves were provided.

The concrete core-wall contained 10,840 cu. yds. and required 13,166 bbls. of cement, or an average of 1.21 bbls. per cubic yard. The grouting and plastering took 2,078 bbls. of cement, in addition. The exact quantity of cement used in grout alone is not known.

Method of Constructing a Thin Cut-off Wall by Grout Injection.

A method of constructing a cut-off wall in a permeable formation which, though used many years ago, is not generally familiar, is described by Col. F. V. Abbot, U. S. Engineer Corps, in Proceedings American Society of Civil Engineers, Vol. XL, p. 1561. The wall was built in connection with the construction of Lock and Dam No. 2, Mississippi River Improvement. At this locality the material forming the bed of the river is a very hard combination of clay, sand and limestone fragments mixed with a few granite boulders and overlying a friable sand rock. The cut-off wall construction is described as follows:

> The seams were all originally water-bearing, and only became dry after long pumping had drained out all the water tributary to them. At a lower level these seams run out into the bed of the river, and in this way passages for water opened between the river bed and the interior of the cofferdam, many feet below the original bottom of the river at the point where the cofferdam stands. It had been realized from the start that such seams were to be encountered, and to avoid opening up too great a flow at any one time a novel form of cut-off wall was adopted after prolonged study. A slot in the sand rock was made, 2 in. wide and 12 ft. long, parallel to the river wall and lying between two longitudinal foundation timbers, by jetting a series of interesting vertical holes 10 ft. deep and breaking off any projecting edges between the holes. Most of the eroded material was brought up out of the slot by the jet, and the rest was easily pumped out with the ordinary Edson diaphragm pump and two men. In the usual case this left a perfectly clean slot with a considerable flow of water pouring out of it, the water coming from the underground seams which connect with the river. Into this slot a diaphragm made of tongued and grooved three-quarter-inch boards, in two levels breaking joints, was lowered by hand. Three vertical 1½-in. iron pipes, extending down into the slot only a few inches, were inserted, one at each end and one at the middle of the slotted section. A steam pump connected to one of these pipes was then started just fast enough to take care of the flow from the slot without letting it run over, or draining it below the level of the surface more than a few inches, and concrete was rammed into the space above the slot and between the foundation timbers above referred to, which had been firmly drift-bolted down to the sand rock before the slot was cut. As soon as the concrete had set—that is, in about 24 hours—the pump was stopped, and water at once flowed out of all three pipes. Additional lengths of pipe were screwed on the three till the flow was stopped—that is, till the tops of the pipes were at a higher level than the head due to the pressure in the seams. Thick grout was now poured into smaller pipes passing through the first pipes and reaching to the bottom of the slot. As the latter filled with grout the water was forced back into the seams. The small pipes choked after the grout rose a couple of feet above their lower ends, and they were then raised till they again flowed freely. When the slot was full they were removed and the larger pipes were kept filled to the top with the grout as long as any more could be poured in. After about 24 hours all the grout had hardened, the pipes were removed, cleaned out, and used over again in another section of cut-off. The perfect grout filling of the most minute cracks in the sand rock was surprising. An absolutely tight cut-off wall from the bank above the lock to the outer toe of the river wall, and under this toe for its full length and back to the bank below the lock, extending to a depth of 23 ft. below dead low water in the river, or deeper in that part under the river wall, was thus secured.
>
> In some cases no subterranean seams were encountered, and then the wooden diaphragm was inserted and the grout at once poured in around it through pipes leading to the bottom. Experience showed that in such cases time was a vital element. A slot which did not develop the least flow at first, in several cases developed very great and troublesome flow in 24 hours if not grouted. Most trouble was encountered at the upper end where a very large seam directly connecting with the river had to be cut off. This seam extended nearly horizontally under a space fully 30 ft. wide by 80 ft. long, and every section of cut-off wall intersected it. Grout poured into one section would pour out of another 30 ft. distant. As these underground connections were not suspected, the quantity of water which poured out of three openings into this seam soon exceeded the capacity of the pumps. All arrangements for grouting two of the three sections were completed except the stopping of the flow from the pipes, which in this case were 6-in. in diameter and were running full. It was believed to be unsafe to try to stop suddenly such a flow for fear of blowing up the lock floor. The cofferdam was allowed to fill, thus gradually stopping the flow from the seam, 6-in. risers were screwed into the now submerged overflow pipes and both sections were grouted at the same time from staging erected above the water level. Twenty-one barrels of dry sand cement were required for the grout to fill the slots and seam. In three days the cofferdam was again pumped out and the two springs were found stopped entirely. The third was really outside of the lock walls, being in a section of slot that had been put in to make connection with the cut-off wall under the dam. It was successfully separated from the cut-off slot under the lock wall, which was afterward easily grouted.

The use of sand cement grout was an unusual feature of this work and tests were made to determine its hardening qualities. Gen. Abbot describes and comments on these tests as follows:

> A vertical glass tube, 4 or 5 ft. long and about 1 in. in diameter, was plugged at the bottom and filled with a 1 to 1 sand-cement grout and allowed to harden. The tube was broken in three days, and it was found that the lower part of the grouting in this pipe was as hard and sound as that at the top. When ordinary sand was mixed in the same proportion with undiluted Saylor's cement, kept thoroughly stirred up, and poured into a similar tube, the sand settled to the bottom very rapidly, leaving a creamy grout with hardly any sand grains in the upper part of the tube. Under these conditions the bottom 2 ft. never set, though the upper 2 or 3 ft. set about as hard as the sand-cement grout did in the other tube. Since making this experiment the writer has always had considerable doubt as to results of grouting with Portland cement stirred up with ordinary sand, but he has no doubt of the effectiveness of grouting with sand-cement ground 1 part of cement to 1 part of silicious sand, provided the grinding is carried far enough to make the sand as fine as ordinary Portland cement.

Tests of Uplift Pressure on Masonry Dams.

To the Editors:—In an editorial on "Neglected Factors in the Designing of Masonry Dams," ENGINEERING AND CONTRACTING, May 27, 1914, the following statement, with reference to uplift pressure in masonry dams, is made: "There is not a test record showing what the intensity and distribution of such pressures may be or indeed showing beyond question that any uplift pressures exist in cases of rock founded dams." Permit me to call your attention to some experiments performed in Germany, the results of which have been reported by R. Schaefer in Zeitschrift für Bauwesen, Vol. 63, 1913, p. 102. These experiments, as far as I know, are the only ones that have been made and show conclusively that, with a certain character of rock foundation, uplift pressure of considerable magnitude may occur, and that the distribution is not always from full static pressure at the heel to zero at the toe, as is commonly assumed.

I heartily endorse your plea for further field investigation along this line, especially since the cost of making such experiments need not be very great.

Very truly yours,
C. R. WEIDNER,

Instructor in Hydraulic Engineering The University of Wisconsin.
Madison, Wis., May 28, 1914.

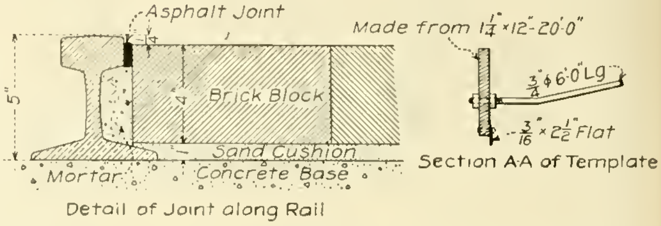
An electric Diesel engine railroad motor car was recently tested in Sweden. It weighs 33 tons and was run at a speed of 34 miles an hour on a grade of 1 in 100. Altogether, 2,200 miles were covered. The consumption of crude oil averaged 9.3 lbs. per mile.

Roads and Streets

Methods and Cost of Laying a Vitrified Block Pavement in Fort Worth, Texas.

Contributed by E. W. Robinson, Civil Engineer with A. J. McKenzie Construction Co., Fort Worth, Texas.

The following data relate to the methods used and unit costs of constructing a vitrified block pavement with cement grout filler on the roadway of the Paddock Viaduct in Fort Worth, Texas. The work was done in April and May, 1914, and during one of the wettest springs ever known in that part of the country. The section paved was 1,741.7 ft. long comprising a reinforced concrete arch and girder bridge for about two-thirds of its length, and an earth fill between concrete retaining walls for the remainder. As shown by the accompanying sketch, Fig. 1, the total width between curbs is 54 ft., but as the paving between the rails of the double car track was not included in this contract the paved area consisted of a space 19 ft. 7 ins. wide next to the curb on each side of the roadway. For about half of the length of the viaduct there is a sidewalk on each side; on the remainder there is nothing but a curb, or wheel guard and railing, on the outer edge of each paved area. The whole structure is on a straight 4 per cent grade, which in case of a heavy rain made drainage difficult on unfinished work. On the other hand, the steep grade allowed water to drain from the sand cushion under the blocks which had not been rolled or grouted, much quicker than would have been the case had the grade been flatter.



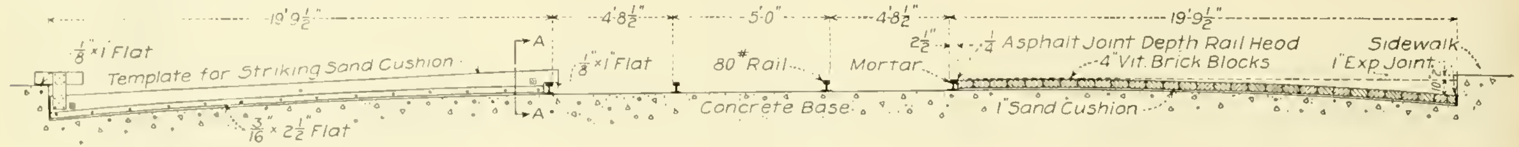


Fig. 1. Cross Section of Vitrified Block Pavement at Ft. Worth, Texas, Showing Details of Striking Template.

MATERIALS AND METHODS.

Specifications called for thoroughly vitrified repressed blocks with rounded edges, to stand a standard rattler test of not over 20 per cent loss and not to absorb more than 2 per cent of their weight on being immersed in water for 48 hrs. The brick company guaranteed them to lay 16 to the square yard. No wheeling or dumping was allowed. On the section of the viaduct where there were sidewalks, the blocks for paving one side were placed in equal piles, half on the walk and half between the rails of the nearest track. On the rest of the viaduct as many as could be placed on the curb were piled there and the rest were placed between the rails of the nearest track. From these piles the blocks were carried to the hackers by men with clamps holding six or seven each. This method proved to be economical for this width of pavement. Where clamps are used the writer has found it pays to see that the blocks are piled carefully in the first place to prevent lost time in getting to them. Where the blocks are piled carelessly, before a man can fill his clamp it is necessary for him to stop and straighten up six brick before the clamp will catch hold of them. The carriers should be able to reach out and grab six blocks, much in the same manner that a man grabs a piece of ice with ice tongs, without hesitating at the pile.

The specifications provided that the surface of the pavement next to the rail should be from ⅛ to ¼ in. below the top of the rail to avoid danger of the car wheels riding on the brick. The web of the rail was plastered with mortar before the blocks were laid in such manner that the blocks would lack ¼ in. in meeting the rail head. This joint for the depth of the head was filled with asphalt so that any vibration of the rail under traffic would not shatter the bond with the rail and allow water to reach the base.

Sand Cushion.—The sand cushion was supposed to be 1 in. thick, but averaged 1½ ins. after rolling, the unevenness of the surface of the concrete slab causing it to vary from ½ in. to 3 ins. in thickness. The sand used from local pits is known locally as dead sand, because of its unfitness for use with cement in mortar or concrete. It has comparatively little clay or loam in it, but is almost a powder in fineness and the grains are round and even in size. It packs well and is excellent for use as a cushion for block pavements. It was hauled onto the concrete base and dumped half a load in a place, spread with shovels, evened up with garden rakes, and rolled with a 250-lb. hand roller. The template shown in Fig. 1 was used to drag the sand to proper grade and contour after which any uneven places were touched up with a lute by a man experienced in that work. It reads well in specifications to state that the sand cushion must be shaped by means of a steel-faced template, but the writer has seen several men who could do the work just as well and considerably cheaper by means of the lute alone. Uneveness in the curbs, or high points in the concrete base, etc., usually keep one man busy most of the time following up with a lute when a template is used. On this work where the loads were dumped at proper intervals three men did all the work on the sand cushion for a brick laying gang laying 35,000 blocks in eight hours. In most instances two men could have done it as well had it not been for the fact that it was necessary to have three to pull the template

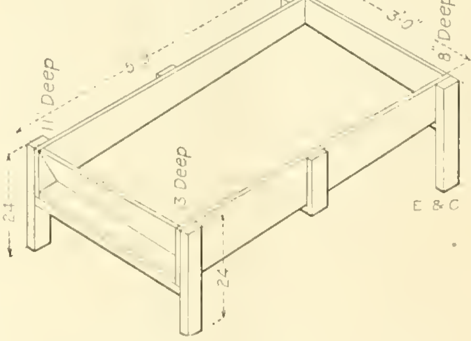


Fig. 2. Persoective of Grout Mixing Box.

Grouting.—The grout for the filler was composed of equal parts of cement and sand and enough water to make it flow properly. An average of 0.0021 bbls. of cement per square yard was used. Two grout boxes, Fig. 2, were used when a full gang was working, and undoubtedly where the work was extensive and the weather permitted fast brick laying, a third box could have been used to advantage. To keep the men around the boxes from standing idle waiting for a batch to be mixed, the cement and sand was mixed dry on the concrete base or on the finished pavement and wheeled to the boxes as fast as needed. This shortened the length of time of mixing in the box and made it about equal to the time it took for the other box to be emptied, thus keeping the whole gang employed. To keep the cement and sand from separating the grout was agitated continuously until the last bit had been dipped out. The first pouring was made thin enough to flow into all cracks and was kept swept ahead by means of steel street brooms. After the first pouring had progressed 25 to 40 ft., depending upon the weather and whether or not the brick were thoroughly dry, one box was turned back to apply the second and final coating. This second pouring was mixed thicker than the first and rubber squeegees were used to fill the joints flush and keep the blocks free from surplus grout. To obtain the ¼-in. joint at the rail it was necessary to fill it flush with the top as with the rest of the pavement and then, before it had set hard to clean the grout out with pointed stick or trowel. Soon after the second coat of grout was placed the whole surface was covered with about ½ in. of damp sand. On dry, hot days the brick were well sprinkled before pouring the grout, considerable water being used for this purpose.

Expansion Joints.—Expansion joints 1 in. wide, were required along each curb and across the pavement at every expansion joint in the floor slab of the bridge. It was first intended to make these joints in the usual manner by leaving a 1-in. x 4-in. strip in place until the grout had been poured and set, and then withdrawing the board and filling with asphalt. Owing to the difficulty usually experienced in removing the strip when cement grout is used, it was decided to use an expansion joint manufactured by the Philip Carey Co. of Cincinnati, O., made in 5-ft. lengths and of the proper thickness and width. This joint has not been in place long enough to tell how efficiently it will care for expansion and contraction under any great range of temperature, but it is undoubtedly convenient to handle and place. In this case there was little difference in the total cost of the two methods, as the saving in labor was offset by the increased cost of material. The labor cost of putting the joint in place is small. Before starting to lay brick one man can string out enough for a day's work in ½ hr., and dropping it in place as the joints are reached does not delay the gang. For the whole job there was used 4,500 lin. ft. of 1-in. x 1-in. joint, weighing 1.65 lbs. per lineal foot, or an average of about 0.6 lin. ft. per square yard of pavement.

Water.—Water for all purposes was brought onto the bridge by means of a ¾-in. hose from two taps, under the bridge, about 200 ft. apart, located about midway of the structure. Sufficient hose was used to cover about 500 lin. ft. of pavement adjacent to the taps. For the rest of the work the water was hauled from the hose in an 8-bbl. tank wagon. Part of the time a team was used to haul the tank back and forth, but as the team often stood idle this did not prove economical. Four men could pull the empty tank up-hill, and three could bring it down loaded. It required six men to pull the loaded tank uphill after starting it to rolling. While this interrupted the gang it proved more economical than a team and much cheaper than a pipe line. It is important to have plenty of water convenient for a grout filled pavement, and getting it to the work may easily be more costly than the water itself. The water used for all purposes amounted to 3 gals. per square yard.

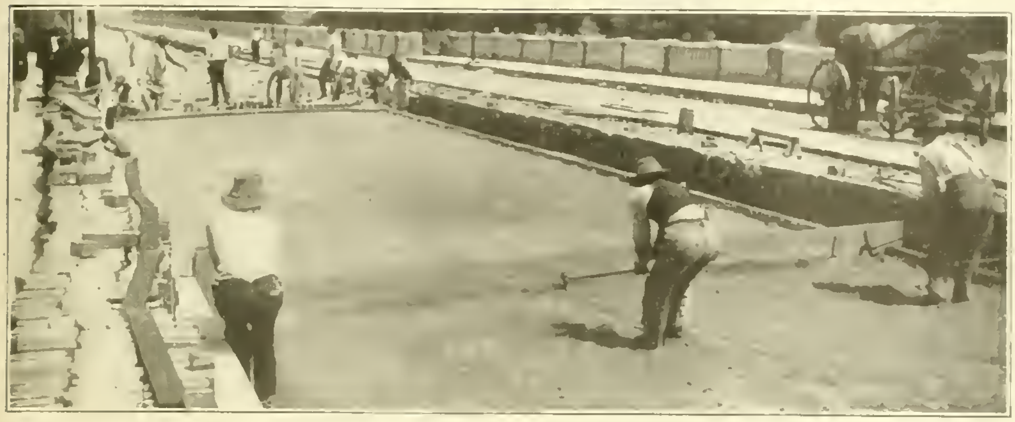


Fig. 3. Method of Striking Sand and Piling and Laying Brick.

COSTS.

Daily cost reports were turned in by the timekeeper, and these had to check the payroll each week to the nearest dollar. Costs were charged to the following items: *General,* including all general labor, such as foreman, timekeeper, night and day watchmen, etc. *Sand Cushion,* all labor, spreading, rolling and shaping after it has been dumped in piles on slab. *Laying Blocks,* laying complete, including carrying, hacking, setting, batting in and straightening courses, etc. *Culling and Cleaning,* getting ready for roller. *Rolling,* labor and repairs, fuel, etc. *Grouting; Expansion Joint,* that along the rail only. *Cleaning Up; Miscellaneous,* including all other items not chargeable to other items.

A 2-ton tandem roller was used, which was found to be just the proper size for this work. However, this particular roller was in poor repair, although the use of it was rent free. The engineman was paid $0.50 per hour for the time he was on the job, whether rolling or waiting to roll. The total cost of rolling was $87, of which $17 was for repairs, $8 for fuel and the remainder for labor.

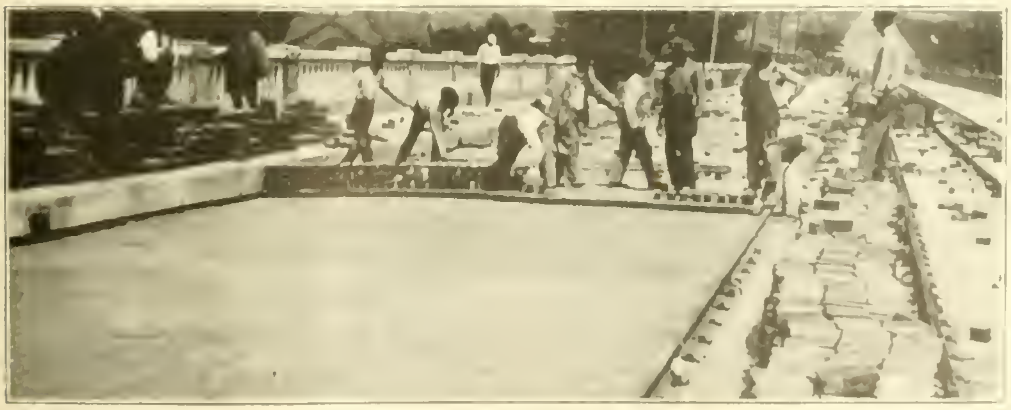


Fig. 4. Typical Brick Laying Gang.

Brick was unloaded by contract except a few cars, and therefore none of that is included in the cost given below. The first contractor unloaded a few cars at $0.75 per thousand, the second man about a third of them at $0.85, but both got tired of their bargains. The average haul was a little over one-fourth mile and part of the road was not very good except in very dry weather. The men had to hustle all day to unload them at an actual cost of $0.75, and neither of the first two men did it at that figure. The third man took the contract to unload the remainder at $0.95 per thousand, which he did at a good profit, as he stayed on the job continuously and saw that every man worked. The prevailing price for work of that kind at this time was $0.25 per hour for men and $0.50 per hour for teams. From 700 to 800 blocks were hauled to a load, which meant a net load of over 6,000 lbs.

Fig. 5. Paddock Viaduct, Ft. Worth, Texas, Paved With Vitrified Block.

Table 1 gives the actual pay-roll with labor costs of the finished work, with the exception of rolling as noted above. These costs do not include any overhead expense, such as superintendence, office, insurance, etc. The general item amounted to 32.2 per cent of the total, and was apportioned to all the other items to get the total.

The general item was higher than it should be on a job of this sort, owing to the many days when no work could be done, as it remained the same with slight exceptions whether work was done or not. Of the total week days available from the beginning to the end of the work, 20 were spent unloading and waiting on material and other unavoidable delays of that nature, 17 were lost on account of rain, and 18 were actually worked with at least part of a gang.

Eight-hour days were worked, as required by law on all public work, and the following typical gang gives an idea of wages paid for labor:

Item.	Total.
Sand cushion—	
8 hrs. at $0.25	$2.00
16 hrs. at $0.20	3.20
Brick laying—	
Setter, 8 hrs at 0.375	3.00
Starter, 8 hrs. at $0.25	2.00
Carrying, 32 hrs. at $0.20	6.40
Hacking, 24 hrs. at $0.20	4.80
Batting in, 8 hrs. at $0.25	2.00
Sweeping and culling, 16 hrs. at $0.20	3.20
Foreman, 8 hrs. at $0.50	4.00
Timekeeper, 8 hrs.	1.50
Night watch	1.70
Total	$33.80

The same men at the same prices were used in grouting and all other work, except that the brick setter was only paid $0.25 per hour when he was not setting blocks.

The record day's run with this size gang was 37,500 blocks, or 815 sq. yds., and it showed that under ordinarily favorable circumstances it could average 35,000 blocks or 761 sq. yds. in 8 hours. This would mean an apparent cost of $0.0411 per square yard for placing the sand cushion and laying blocks ready for the roller, against an actual cost given above of $0.072 per square yard. The difference can be accounted for in many ways. In the first place there were very few days when conditions were favorable for good work and the gang could not begin to make the run of 35,000 blocks. On very windy days the fine cushion sand would almost blind the men at times, as it would blow about like cement. Rain interrupted the work so often that nearly every start made was with several green men in the gang who had to become accustomed to the work. Starting and stopping the work amounts to considerable when only an hour or so of actual work is done each time. In general a large part of the difference was

TABLE I.—UNIT LABOR COSTS OF LAYING 7,660 SQ. YDS. OF VITRIFIED BLOCK AT FORT WORTH, TEXAS

Item.	Cost per sq. yd.
Sand cushion, 3 men	$0.013
Laying blocks, 10 men, average	0.046
Culling and cleaning, 1 to 4 men	0.013
Rolling	0.012
Grouting, 12 to 15 men	0.017
Expansion joint, 2 men, odd times	0.002
Cleaning up	0.005
Miscellaneous	0.008
Total labor cost of laying brick	$0.116

caused by unfavorable conditions under which the work was prosecuted, but even had conditions been favorable there would have been considerable difference.

Experience teaches the difference in apparent cost of doing work when you are paying for it yourself and when you are keeping the cost data on some one else's work. The inspector or engineer is able to get valuable cost data, but from the fact that he is quite apt to be absent when no work is going on he will miss many of the numerous small expenses the contractor is subject to continuously and which in the long run amount to no mean percentage of the total. Figuring cost of work by typical organization and unit performances under favorable conditions is an excellent check against other methods of securing data, but should be used advisedly.

Brenneke & Fay, Consulting Engineers, St. Louis, Mo., were the engineers for the county and the work was done by the A. J. McKenzie Construction Co. under the immediate supervision of the writer.

Asphalt and Bituminous Concrete Pavements in Washington, D. C.

The city of Washington is noted for the excellence of its bituminous pavements. The information given here was presented in part by Captain Mark Brooke in the Proceedings of the American Society of Civil Engineers and in part compiled from data furnished by Captain Brooke to the editors.

In Washington, as a result of experience and of consideration of the various factors such as first cost, character and volume of traffic, suitability of local material, esthetic considerations, etc., the problem of securing the best type of pavement has been reduced to a selection of one of three types of asphalt pavement for city streets; and bituminous concrete, cement concrete, or water-bound macadam with a surface treatment of light tar or oil, for suburban streets and county roads.

In the city practically nothing is used but a standard sheet-asphalt, a very limited quantity of 2-in. asphalt block, and an increasing quantity of bituminous concrete in accordance with the specification of the department. This bituminous concrete, which is very similar to the various pavements of that type laid between 1870 and 1880, appears to have all the advantages and but few of the defects of sheet-asphalt, and is cheaper.

A gravel cement concrete base is now being used under all pavements, except in a few instances where old macadam roads have been surfaced with bituminous concrete. Macadam, especially an old macadam road, will make an excellent base which will outlast any bituminous wearing surface yet devised, but it cannot be kept free from local settlements, nor can it be easily or economically resurfaced. The man who lays bituminous concrete on a macadam base should realize that he is storing up trouble for himself or his successor 15 years hence.

Whether or not it is advisable to invest in the more expensive base to reduce the cost of future resurfacing is, of course, a question every one must decide for himself. The engineers in Washington for some time past have been of the opinion that it is better, as a general practice, to lay a base which will not have to be destroyed every time a new surface is put on it, and hope to relieve their successors of the extra expense and difficulties encountered during the last ten years in resurfacing pavements laid on macadam and bituminous macadam bases—types of construction which conformed to the best engineering practice of the time, when the use of Portland cement was still in its infancy.

For 30 years a concrete base has been laid without either longitudinal or transverse joints, with no significant bad results, and the writer doubts very much the necessity of the elaborate and costly provisions for such construction which are so often seen in cement road work. On some of the cement pavements in Washington, no joints whatever have been provided, on others there is a thin transverse joint every 50 ft., made by bringing the concrete up to a vertical face, laying a strip of folded tar paper against this face, and then depositing the next batches against the paper. Thus far no difference in the action of the two classes of construction has been observed. If any joint is used it should be as thin as possible. A wide joint is a weak joint, and its partial contraction will invariably force the filler into a pronounced ridge, as objectionable to motor traffic as any thank-you-ma'am.

When this pavement is laid with a curb it is kept about 1 in. below grade, so that, should the thin bituminous carpet with which it is covered prove an uneconomical form of wearing surface, the pavement can be used as the base for a thicker surface of bituminous concrete and still have sufficient face of curb showing.

For many years trap rock has been the standard material for water-bound macadam. Recent experiences in the oil and tar surface treatment of trap and limestone roads indicate that, for moderate traffic at least, the surface treatment does as well with the limestone as with the trap, and the saving effected by using limestone about offsets the first cost of the surface treatment.

The practice of paving the center 8 ft. of certain roads with concrete and providing wide macadam shoulders for the light traffic the writer believes is a mistake unless the paved strip is made wide enough to carry two lines of traffic. If this is done, it is unnecessary for passing vehicles to cross the joint between the concrete and the shoulders—the weakest part of the road—a condition which would be particularly objectionable at seasons when the earth shoulders were unsuitable for traffic, and all vehicles would seek the paved portion of the road. The engineers in Washington have had this experience in the case of a trunk-line road paved with granite block, the center 8 ft. of which was laid with asphalt for the benefit of light traffic. As a matter of fact, all traffic, both slow and fast, tries to get on and stay on this 8-ft. strip, thus increasing both the congestion of traffic and the wear on the asphalt.

The specifications for asphalt wearing surface and bituminous concrete are given below:

ASPHALT WEARING SURFACE.

The wearing surface of the pavement shall be composed of asphalt, petroleum oil, asphalt cement, clean, sharp-grained sand, fine absorbent mineral dust.

Asphalt.—The asphalt shall be refined until homogeneous and free from water and shall not at any time be heated to a temperature high enough to injure it. The refined product shall contain at least 90 per cent of bitumen soluble in carbon bisulphide and 100 parts shall not require more than 80 parts of the flux to produce the asphalt cement described under asphalt cement.

Petroleum Oil.—The oil in use in the manufacture of asphalt cement shall be a petroleum from which the lighter oils have been removed by distillation without cracking, until the oil has the following characteristics: Free from water and foreign matter; flash point, not less than 300° Fahrenheit; distillate at 400° for 18 hours, less than 10 per cent; the flash point shall be taken in a New York State closed oil tester. The distillate shall be made with about 50 grams of oil in a small glass retort, provided with a thermometer and placed in a copper holder. The residue in the retort, after distilling, must be fluid at 75° F., and not coarsely crystalline on cooling.

Any other softening agents approved by the Engineer Commissioner may be used in place of petroleum oil.

Asphalt Cement.—The asphalt cement must be of refined asphalt, fluxed when necessary with petroleum residuum, asphaltic oil, refined maltha, or other approved flux. The cement must be practically free from water and must be within the range of 40 and 70 penetration when tested at 77° F. on Dow penetration machine with No. 2 needle, 100 grams, 5 seconds. The degree of penetration to be fixed by the Engineer Commissioner.

Preference will be given to an asphalt cement that is not readily affected by the action of water, provided it is satisfactory in other respects. If an asphalt cement is accepted that is affected by water some provision satisfactory to the Engineer Commissioner must be made to guard against the results of such action, and such work must be included in the price bid. The use of an asphalt under these specifications shall be subject to the approval of the Engineer Commissioner, and if an asphalt has been proposed for use by the contractor and approved by the Engineer Commissioner no change in the asphalt to be used shall be made unless with the approval of the Engineer Commissioner. If an asphalt or flux is submitted for use which has not been successfully used for a period of at least two years for paving under conditions similar to those existing in the District of Columbia, its use may be limited to such extent as may be deemed advisable, or it may be rejected for use entirely in the discretion of the Engineer Commissioner.

The bitumen of the asphalt cement must comply with the following tests:

1. It must be of such consistency that when tested at 32° F. it will not show a hardness below 10 penetration, and when tested at 115° F. it will not be softer than 350 penetration. 2. When a briquette of the bitumen having a minimum cross section of one square centimeter, having a penetration of 50°-53° at 77° F. is tested for ductility at 77° F., the bitumen must stretch at the rate of 5 centimeters per minute to a distance of 20 centimeters before breaking. 3. When the bitumen is heated in an open tin box ¾ ins. deep by 2½ ins. in diameter at a temperature of 300° F. for eighteen hours in a hot-air oven it must not show a loss by volatilization of over 5 per cent and it must not have been hardened over 50 per cent by this heating.

The asphalt cement must never be heated to a temperature that will injure it.

When the asphalt cement contains over 5 per cent of material that will separate by subsidence while in a molten condition it must be thoroughly agitated before drawing from storage and while in use in the supply kettles so as to insure a uniform cement.

These properties shall be determined by tests made by uniform methods, descriptions of which are on file in the office of the Engineer Commissioner.

Sand.—The sand in use shall be free from mud, hard-grained and moderately sharp. On sifting it should have at least 15 per cent of material that would be caught on a 40-mesh per inch screen, 25 per cent of material that will pass an 80-mesh to the inch screen, and 10 per cent at least must pass a 100-mesh to the inch screen. If the sand to be used does not contain the desired fine material, mineral dust can be added to make up the deficiency, and in any case at least 5 per cent of such mineral dust shall be used. The amount of fine material may be increased at the discretion of the Engineer Commissioner.

Mineral Dust.—This shall be any fine Portland cement or limestone dust, the whole of which shall pass a 30-mesh screen, and at least 85 per cent pass a 100-mesh screen.

Asphalt Paving Mixture.—The materials complying with the above specifications shall be mixed in proportion by weight, depending upon their character and the traffic on the street, and upon the character of the asphalt, and will be determined by the Engineer Commissioner, but the percentage of bitumen in any mixture soluble in carbon bisulphide shall not exceed the limits, 9 to 13 per cent. If the proportions of the mixture are varied in any manner from those specified the mixture will be condemned; its use will not be permitted; and, if already placed on the streets, it must be removed and replaced by proper materials at the expense of the contractor.

The sand or the mixture of sand and stone dust, and the asphalt cement, will be heated separately to about 300 degrees Fahrenheit. The dust, if limestone, will be mixed while cold with the hot sand in the required proportions and then mixed with the asphalt cement at the required temperature, and in the proper proportion in a suitable apparatus, so as to effect a thoroughly homogeneous mixture. Sand boxes and asphalt gauges will be weighed in the presence of inspectors as often as may be desired.

Samples of all material entering into the composition of the pavement shall be supplied to the Inspector of Asphalt and Cements when required, in suitable tin boxes and cans; he shall have access to all branches of the works at any time, and shall have the right to obtain samples of all materials from the source of supply.

Laying Asphalt Surface.—The pavement prepared in a manner thus indicated will be brought to the ground in carts or wagons at a temperature of not less than 250° or more than 350° Fahrenheit; the contractor must provide canvas covers for use in transit. It will then be shoveled into place and thoroughly spread to a thickness of at least 2½ ins. by means of hot iron rakes, in such manner as to give uniform and regular grade, so that, after having received its ultimate compression, it will have a net thickness of at least 1½ ins. This depth will be constantly tested by means of gauges furnished by the Engineer Commissioner. The surface will then be compressed by steam rollers, after which a small amount of hydraulic cement will be swept over it, and it will then be thoroughly compressed by a steam roller weighing not less than 175 pounds to the inch run, the rolling being continued for not less than five hours for every 1,000 yards of surface. The street to be barricaded. Binder or topping shall not be laid when weather conditions are, in the judgment of the engineer, unsuitable for the work of laying the pavement. Barricades to remain for such length of time as deemed necessary by the Engineer Commissioner.

Asphaltic Base.—Asphaltic base will be composed of clean broken stone, free from spalls, that will pass through a two-inch ring, well rammed, and rolled with a steam roller weighing not less than five tons. The rolling will be continued until the stone ceases to creep before the roller, and it is evident that the final compression has been reached. It will then be thoroughly coated with asphaltic paving cement of approved quality, as directed.

ASPHALTIC CONCRETE ON CONCRETE BASE.

Paving materials.—The paving materials shall be composed of crushed trap rock screenings, concrete sand and mineral dust in the following proportions: Trap rock screenings, 2 parts; concrete sand, 1 part; and mineral dust, at least 5 per cent of the above aggregate; mixed with asphalt cement.

Trap Rock.—The trap rock shall be of a quality to be approved by the engineer, and shall be equal to that used by the District of Columbia for macadam roadways. The crushed stone will vary in size from 1 inch to screenings and shall be devoid of dust.

Sand.—The sand shall be hard-grained and moderately sharp. On shifting it should have at least 25 per cent of material that would be caught on a 20-mesh per inch screen, and 5 per cent of material that will pass an 80-mesh to the inch screen. If the sand to be used does not contain the desired fine material, mineral dust can be added to make up the deficiency, and in any case at least 5 per cent of such mineral dust shall be used.

Asphaltic Concrete Paving Mixture.—The materials complying with the above specifications shall be mixed in proportions by volume depending upon their character and the traffic on the street, and upon the character of the asphalt, and will be determined by the Engineer Commissioner, but the percentage of bitumen in any mixture soluble in carbon bisulphide shall not exceed the limits, 7 to 9 per cent. If the proportions of the mixture are varied in any manner from those specified the mixture will be condemned; its use will not be permitted; and, if already placed on the streets, it must be removed and replaced by proper materials at the expense of the contractor.

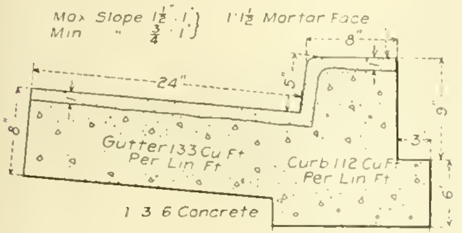


Fig. 1. Combined Curb and Gutter Used in Washington, D. C.

Laying Asphaltic Concrete Surface.—The stone and paving cement shall be heated separately to a temperature of about 300°, and shall be thoroughly mixed while hot by machinery. The proportion of paving cement shall be sufficient to thoroughly coat each particle of the aggregate, and the entire mixture shall be subject to the approval of the engineer. The mixture will be hauled while hot to the site of the work and shall be covered until deposited on the street. The temperature at the time of dumping shall not be less than 220°. The hot mixture shall be evenly spread with hot tools upon the base to such a thickness as will make a layer 2 ins. in thickness after rolling. It shall then be rolled with a steam roller weighing not less than one ton per foot of tread of roller, until no further compression occurs. After the rolling of the asphaltic concrete wearing surface has been completed there shall be spread over such surface a thin coating of asphaltic cement as used in surface not to exceed on an average a quarter of a gallon to the square yard, of such consistency as shall be approved, which shall be thoroughly brushed into the wearing surface so as to fill all voids and smooth out any minor unevenness of the said surface. There shall then be spread over and rolled into this flush coat a thin layer of trap screenings, so far as practicable, devoid of dust, in size from ¾ in. down, whose use shall be to the end of securing a gritty, no-slippery surface. The finished surface shall be free from lumps or depressions and shall be true to the required cross-section. The street to be barricaded. Barricades to remain for such length of time as deemed necessary by the Engineer Commissioner.

Asphaltic Concrete on Broken Stone Base.—A surface coat of asphaltic concrete complying in all respects to the specifications for asphaltic concrete on concrete base is to be laid on a base of broken stone or gravel. The base will be furnished by the District of Columbia, in place and rolled, ready for surfacing. The price bid will include supplying, mixing, placing and rolling the asphaltic concrete surface.

Resurfacing Over Asphalt and Coal-Tar Pavements.—The foregoing specifications shall also apply, as far as practicable, to all work of resurfacing. Where the binder coat cannot be made of uniform thickness, it will be paid for by the cubic foot.

The dedication of the new filtration plant at Cleveland, Ohio, will take place in the fall of 1915, simultaneously with the celebration of the 100th anniversary of the incorporation of the village of Cleveland.

Curb and Gutter Construction in Washington, D. C.

(Staff Article.)

The curb and gutter construction used in Washington, D. C., is of an unusually substantial type. It is a rare thing to see curb out of line or a broken gutter in that city. In Fig. 1 is shown the standard type of combined curb and gutter in general use. It will be noted that the curb is unusually thick and the base therefor is quite broad. The method employed in laying is as follows:

Concrete Curb and Gutter.—A trench is excavated and trimmed to a depth of at least 14 ins. and a bed of bank gravel, free from excess clay or loam, spread to a depth of 6 ins. and rammed. In making the curb, forms are placed and the mortar face formed by plastering on the inside before the curb concrete is laid. Curb and gutter is divided in 10 ft. lengths, a clean cut joint being made either by use of a thin metal plate or with heavy paper. The face forms on the curb are removed within 24 hours and the surface troweled to a neat finish.

The specifications call for a high quality of sand and no stone over 2 ins. in diameter. The mortar surface on gutters is placed immediately after laying concrete and thoroughly manipulated by troweling and beating with wooden battens so as to break any air cells and make the surfacing solid. Joints are filled with sand. A coating of dry cement and fine sand, 1:1 mix, containing thoroughly incorporated coloring matter is floated on the surface. A jointing tool is used cutting to a depth of ½ in. and the exposed surface covered with sand for protection.

Granite Curb and Brick Gutters.—The granite curb and vitrified block gutter is shown in Fig. 2. Granite curb is set in the following manner:

A trench is excavated 18 ins. wide and 15 ins. below the top of the curb when set. One edge of this trench is 1 ins. beyond the finished curb line toward the center of the street. Five inches of 1:4:10 pebble concrete is placed in this trench and the curb set immediately, heavy mauls being used to bed it firmly. As soon as the curb is laid the trench on the sidewalk side is filled to within 5 ins. of the top of the curb with concrete tamped in place. Excess concrete in front of the curb which will interfere with laying vitrified block gutters is removed and earth filled in behind the curb to prevent too rapid drying out of the concrete. No provision is made for longitudinal joints between the curb and the street surface base.

In laying the gutter as soon as practicable after the concrete base has been laid, a dry mixture of sand and cement is spread to a thickness of not less than ½ in. as a bed for the paving blocks. The blocks are laid on edge joints being broken so that each block has at least a 4-in. lap in every course. A plank is laid over several courses and a rammer used until the blocks reach a firm bed and present a uniform surface. After ramming the gutter is grouted with a thin, easy flowing grout of neat cement.

The following are the contract prices for concrete curb and gutter in 1914:

Item.	Cost per lin. ft.
Curb alone	$0.52
18-in. gutter	0.31
24-in. gutter	0.42

We are indebted to Capt. Mark Brooke, Engineer Corps, U. S. A., and assistant to the engineer commissioner of the District of Columbia for the information contained herein.

The Use of Hydrated Lime in Concrete Road Surfaces.

The problem of reducing the amount of water which leaches from a newly laid concrete road surface is a serious one. Several pallatives have been proposed, among them the addition of hydrated lime to the concrete. Whether the strength and hardness are affected to the extent of overbalancing the advantages derived from the increased water-tightness of the concrete has not been determined. The advantages accruing from the use of this material are pointed out in a paper by Robt. S. Edwards, before the National Lime Manufacturers Association, which is given here in part.

The general conditions under which concrete pavements are placed are severe, as the majority of work is usually done in the dry spring and summer months when temperatures are high, causing excessive evaporation. The concrete is generally poured on an earth or clay sub-base in a thin slab varying between 4 ins. and 7 ins. in thickness. This condition alone makes its almost impossible to secure uniformity throughout the concrete mass, so that after hardening the concrete road slab is subject to far greater variation, due to the segregation of the coarse aggregate from the mortar, than is found in other forms of concrete work. This, of necessity, causes unevenness throughout the concrete, and the excessive cracking and unraveling so often seen in poorly laid concrete roads is directly due to this fact.

It is the writer's opinion that the primary cause of cracking in concrete pavements is due to the loss or leakage of varying portions of the mixing water from the concrete by seepage through the sub-base, as well as by evaporation from the surface of the concrete, which leaves the concrete road slab in such a condition that at later times it will alternately absorb and give up water, depending on the season of the year, and that expansion and contraction of concrete and any movement in its volume are due infinitely more to this alternate wet and dry condition than to any temperature change from hot to cold without presence of moisture.

That the cracking which is directly attributed to the change in the condition of the sub-base, either in settlement spots or in drying out in shrinking, is primarily due to this cause, which is greatly helped by unevenness and variability of the concrete mixture.

That there are extremely few, if any, concrete pavements, individual slabs of which can be considered entirely uniform.

The Use of Hydrated Lime.—Hydrated lime is a product formed by the addition of accurate quantities of water to known weights of freshly burned quicklime, the finished material being a flour-like powder of great fineness and covering capacity, having more than twice the volume of the same weight of cement in its dry state.

The use of hydrated lime as a void filler in concrete mixtures, and for rendering concrete mixtures more water-tight, dates back several years. Numerous tests show that a replacement of from 10 to 15 per cent of cement by hydrated lime in rich concrete mixtures does not decrease the compressive strength of the concrete after three to four months' time, and ultimately increases the strength and permanence of the concrete. It apparently makes but little difference as to its action as a void filler, whether the hydrate is manufactured from a dolomitic or high magnesia lime or a high calcium lime—it being necessary only to have a product which contains no free lime or unslacked particles. The proper hydration or dry slacking of lime reduces it to a finer degree of subdivision than can be accomplished by any mechanical means.

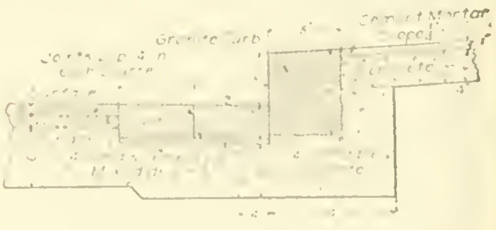


Fig. 2. Granite Curb and Vitrified Block Gutter Used in Washington, D. C

The amount of water required to convert the dry hydrated lime into a plastic paste varies from 50 to 70 per cent of the weight of the lime; while, to obtain a mortar of similar consistency with Portland cement, an addition of 25 to 30 per cent of water is required.

Unlike Portland cement, however, the addition of water to hydrated lime does not create any chemical reactions. There is simply a lime paste formed which holds its excess water for a long time. Upon the addition of small quantities of hydrated lime, amounting to 10 or 15 per cent, to the weight of cement used in concrete mixtures and the ordinary mixing of such mixtures, the hydrate, which has absorbed a large amount of water, corresponding to its weight, is thoroughly disseminated throughout the concrete mass, and, due to its fine plastic conditions, it is forced immediately into places in the concrete which the cement cannot reach.

Concrete, so mixed, on leaving the mixer, flows more readily into place and the coarse aggregate has much less tendency towards segregation in the concrete mass. In other words, the extremely finely divided condition of the moistened hydrated lime paste has increased, to a marked degree, the covering or spreading quality of the cement, and reduced, to a similarly marked degree, the friction between particles of fine and coarse aggregate.

This answers the question "Why does hydrated lime benefit concrete?" The action is purely a mechanical one, and can be compared to the oiling or greasing of a shaft or bearing. The property which hydrated lime has of absorbing and retaining large percentages of water offers a ready solution to the problem of finding a means for keeping and holding moisture in freshly poured concrete pavements until the slower-acting cement-hardening process can utilize it.

It is obvious that small percentages of hydrated lime, when added to concrete mixtures, in road work, will accomplish the following things:

1. Renders the concrete highly plastic and homogeneous, thus producing density and uniformity in the finished concrete. This condition adds much to the life and efficiency of the road because of the uniform resistance to wear and other stresses.
2. Keeps a certain amount of excess moisture in the concrete while setting.
3. Renders the concrete mass more nearly water-tight, thereby preventing the alternate wetting and drying out of the finished concrete.
4. In large pieces of work, these properties given to concrete by hydrated lime would materially reduce the labor in spreading and bringing the concrete to a uniform surface, which is of great importance in paving work.

Road Dragging System in Allamakee County, Iowa.

(Staff Article.)

The system of earth road maintenance instituted in Iowa in 1913 requires that all graded earth roads be dragged or otherwise maintained. The roads comprising the county system (about 15 per cent of the total mileage) are under the direct charge of the county engineer and the remainder under superintendents appointed by the trustees of the various townships. A definite fund for this work is provided by a non-divertable one mill tax.

The method of maintenance adopted by Allamakee County and the various townships therein is based upon individual contracts made with residents adjacent to the road to be maintained. These contracts provide for the operation of a road drag at a definite rate per mile traveled, the drag to be used when instructed by the engineer or superintendent, as the case may be. This method is in general use throughout the state.

In this county the roads are divided into sections six miles long and a contract entered into with a bonded contractor on each section. As a rule one man does not care for more than three miles of road. Upon the completion of a dragging the contractor is required to make a report on a postcard, the form of which is shown in Fig. 1. The average contract price for 'this work is $0.43 per mile. The form of contract in use is given below.

This dragging work is, for the most part, purely maintenance. Repair and minor construction work are generally carried on by force account gangs, under the direction of the township superintendent and are paid for out of the township road funds which are separate from the dragging fund.

The forms and data given herein were furnished by W. H. Miner, county engineer of Allamakee County, Iowa, who has general charge of the work in that county.

ALLAMAKEE COUNTY CONTRACT TO KING-DRAG.

This agreement made and entered into this .... day of ...., A. D. 19...., by and between Allamakee County, State of Iowa, by ...., member of its Board of Supervisors, party of the first part, and ...., party of the second part, witnesseth:

That for and in consideration of the labor, acts and covenants hereinafter mentioned, to be performed by the second party, the first party hereby agrees to pay to the second party, at the times hereinafter mentioned, the sum of .... cents per mile for each mile actually traveled in King-dragging that portion of the county highway as set out below, and removing the loose rocks and stones therefrom;

That for and in consideration of the sum of money above specified, the second party hereby agrees to King-drag the following section .... of the county highway in said county and remove therefrom all loose rocks and stones, especially such rocks and stones as may be loosened by the King-drag, the said section described as follows, to-wit: ....
....

CONTRACTOR'S REPORT FOR ROAD DRAGGING.

County Road System, Allamakee County, Iowa.

I received notice from D. D. RONAN to drag Section No. ...... of the .... Road on the .... day of ...., 191.., at .... o'clock .... m., and did on the .... day of ...., 191.... comply with the same.

Miles dragged, ...; price per mile, $..........; amount charged, $.......... If not dragged, why not?........................
....
........................., Contractor.

To be mailed to the Supervisor within 24 hours after each dragging is completed.

Fig. 1. Form of Dragging Report Card Used in Allamakee County, Iowa.

It is specifically agreed and understood that the said second party shall King-drag said highway and remove the loose rocks and stones therefrom within twenty-four (24) hours after receiving notice to King-drag from Supervisor of said district;

The said Supervisor shall notify second party of the time to King-drag and said second party shall King-drag and remove rocks and stones as above finished, giving the date and number of miles set out, and shall also fill out one of the cards, that will be furnished him by the first party for that purpose, immediately after said King-dragging is dragged, and mail said card to said ...., Supervisor.

Payment by first party to second party shall be made on the 10th day of each and every month for all King-dragging done by second party during the month preceding as shown by cards received by said ...., Supervisor; the intention being to allow the first party 10 days to make computation and mail warrant for amount due second party after close of preceding calendar month.

In witness whereof the parties hereto have hereunto set their hands and seals this ...... day of ...., A. D. 19....

ALLAMAKEE COUNTY.
By.........................
First Party.
By.........................
Second Party.

Equipment Used in Constructing 12 Miles of Concrete Roads in 8 Months.

(Staff Article.)

This equipment is an inventory list of the field and office appliances used in constructing 12 miles of concrete road by force account in one working season. The list was furnished us through the courtesy of Mr. H. J. Knelling, highway commissioner of Milwaukee county, Wisconsin. Some duplications, of course, exist, since there are times when the purchase of extra equipment is necessary to avoid expensive delay and it is hardly fair to charge them to cost of equipment, although this is the universal practice.

From ten to fifteen separate gangs were operated, including surfacing, hauling, culvert and bridge, gravel pit, tarring and grading. In addition the surveying equipment and the testing outfit for testing cement, bituminous and concrete aggregate are given.

CONCRETE MACHINERY.

Article.	Cost.
635 ft. 1½-in. water hose	$ 214.31
645 ft ¾-in. water hose	87.08
15 ft. ¾-in. syphon hose	16.80
20 ft. 2 -in. suphon hose	9.00
20 ft. 2½-in. syphon hose	21.00
24 ft. ¾-in. steam hose	10.44
54 ft. 1½-in. steam hose, new	28.08
8 ft. 1½-in. steam hose	4.80
Clutch for Milwaukee mixer	2.00
1 ½-in. Penberthy injector	6.50
1 1-in. throttle valve	1.52
1 hand lubricator	1.25
1 1-in. stop cock	.74
1 ¾-in. barret faucet	.55
1 steam gauge	1.25
15 ft. 2½-in. suction hose	8.02
56 ft. 2 -in. suction hose	22.10
50 ft. 2½-in. suction hose	26.72
14 ft. 1 -in. steam hose	7.49
1 Milwaukee mixer No. 14	
25 ft. ¾-in. water hose	4.00
1 chain belt mixer	
1 rotary water pump	6.75
2 ft. 4-in. belt	.65
1 Chicago mixer No. 10	
1 Koehring mixer No. 14	
36 wheelbarrows	135.00
20 ft. 1 -in. water pipe	1.60
20 ft. 1 -in. steam hose	10.00
20 ft. 2 -in. steam hose	12.00
40 ft. 1½-in. water hose	13.43
20 ft. ¾-in. water hose	2.67
6 canvas covers	144.00
22 canvas covers	362.56
1 diaphragm pump	39.00
1 Milwaukee mixer (culvert work)	
1 Koehring bar bender	63.70
1 Koehring bar cutter	21.56
270 iron stakes	34.43
10 pair rubber boots	39.59
2 suction hose sieves	1.50
1 hose reel	1.00
1 wheelbarrow wheel	1.00
1 1½-in. globe valve	.92
2 lubricators	5.00
1 new countershaft gear for Chicago mixer, with dia. clutch	7.80
1 new clutch ring, Milwaukee mixer	2.00
1 ½-in. Penberthy injector	6.50
6 pair boots	22.75
2 trowels	2.50
1 ½-in. Jenkins valve	.40
2 spark plugs	2.00
4 wheelbarrows	16.00
10 water gauge glasses	.60
65 lbs. No. 12 wire	1.95
1 Detroit lubricator, 1-pt	4.28
1 brass hand oiler	1.25
1 steam whistle and valve	2.25
Total for concrete machinery	$6.780.02

HAULING MACHINERY.

Article.	Cost.
2 kegs railroad spikes	$ 10.60
1 keg railroad bolts	9.75
1 1 1-3-in. steam throttle valve	1.75
3 traction spindles, new	15.00
1 steel wagon axle	12.00
12 ft. ¾-in. chain, hook and link	5.50
5 ft. ¾-in. chain, hook and link	2.00
1 Reeves tractor	
5 ⅜-in. comp. grease cups	1.25
1 flue cleaner	.72
6 calks for drive wheels	
1 Case tractor	
2 draw bars	13.00
6 mud calks	
2 exhaust nozzles	1.50
12 ft. ¾-in. chain, hook and link	2.90
1 flue cleaner	.72
1 Avery engine	
2 Case tanks with wagons and pump	222.35
1 Reeves tank with wagon and pump	100.00
1 Russell tank with wagon and pump	114.15
3 Austin 2-yd. dump wagons	345.00
5 Troy 2-yd. dump wagons	527.76
12 ft. ¾-in. chain, hook and links	5.75
5 ft. ¾-in. chain, hook and links	1.45
12 ft. ½-in. chain, hook and links	3.25
12 3½ Troy wagons	
1 Avery push pole	2.00
1 ash hoe	.75

6 pokers.	3.00
1 rake.	.75
1 draw bar.	2.50
2 wagon couplings.	3.00
4 steering pins.	.80
4 coupling pins.	.80
1 clutch clamp, new.	.60
1 soft plug for tractor	.80
2 steam gauges.	2.50
12 mud calks.	1.50
2 new spindle washers.	.30
1 Avery grate.	2.00
1 Avery driving pinion, new	3.40
30 ft. ½-in. chain	8.10
1 Lindsay tank outfit	116.25
2 wagon spindle patterns.	32.00
1 grease cup.	.10
1 pattern for car bearings	5.00
40 ft. ¾-in. rope	1.00
Total for hauling machinery	$13,456.01

GRADING MACHINERY.

Article.	Cost.
23 Western wheeled scrapers.	$ 940.12
8 eveners, made by county	9.60
17 singletrees.	20.40
18 eveners with singletrees	64.80
2 steel eveners with singletrees	7.20
2 neckyokes.	2.40
4 Case wheel scrapers, No. 2½	158.10
8 Good Roads wheel scrapers	320.00
1 Road King grader.	405.00
3 No. 2 Dorsch plows	68.00
2 Western pick plows.	60.00
21 slip scrapers.	95.00
1 Dorsch plow No. 2 (bent beam)	24.00
1 farm plow.	10.00
2 eveners with singletrees (4)	7.20
1 pick plow point	2.50
1 chain with snatch	.60
3 singletree hooks.	.75
12 clevises.	4.20
2 doubletree rings.	.60
3 plow shoes.	4.50
1 plow point.	3.00
1 slip scraper.	5.00
1 Giant grader.	250.00
Total for grading machinery	$2,462.97

PUMPING MACHINERY.

Article.	Cost.
1 Fairbanks pump, engine, etc	$ 140.00
1 box batteries for gas engine	1.50
1 Columbia Ex. battery.	3.00
1 socket wrench.	.40
2 valve springs for F. M. engine	.30
4 2-in. galvanized couplings	.80
1 carburetor for Lauson engine	6.00
1 4-h.p. Lauson engine and Myers pump complete.	296.00
1 4-h.p. Lauson engine and Myers pump complete.	296.00
1 switch and coil	5.50
1 induction motor, 5-h.p.	123.25
1 sprocket wheel for pump	3.00
3 14x4 pulleys for pump	12.60
2 ft. drive chain.	.23
10 ft. 4-in. leather belting	3.25
7 2-in. brass valves.	14.21
25 lbs. broken brass valves	
1 14-in. chain sprocket for Myers pump.	3.00
35 2-in. plugs.	1.17
2 new crossheads for Myers pump	7.00
30 ft. 6-in. 1-ply rubber belting	5.49
1 steam flue door	5.50
1 2-in. brass union.	.89
50 2-in. pipe couplings	2.00
15 ft. chain drive, new	10.31
1 5x7 Myers pump and house	65.00
15 ft. 2-in. suction hose	13.50
10 ft. ¾-in. water hose	1.34
25 lbs. brass scrap	
1 2½x2-in. hydrant reducer.	1.75
3,117 lengths of 2-in. pipe	5,676.06
64 2-in. unions.	51.74
18 2-in. valves.	38.34
73 2-in. T's.	18.85
29 2-in. L's.	5.33
1 2-in. foot valve.	.47
2 2-in. check valves.	6.25
Total for pumping machinery	$6,820.03

OILS, GREASES AND PACKING

Article.	Cost.
½ barrel steam cylinder oil.	$17.68
1 empty oil barrel.	1.00
½ barrel lubricating oil.	7.50
⅓ barrel asphaltum oil.	3.00
¼ barrel gas engine oil.	5.03
¼ barrel steam cylinder oil.	6.75
1 empty half barrel.	1.00
100 lbs. grease.	12.00
3 2-gal. oil cans.	1.05
4 1-gal. oil cans.	.70
5 1-gal. oil cans.	.90
3 5-gal. oil cans.	1.50
2 3-gal. oil cans.	.80
12 sq. ft. of packing	7.20
2 lbs. Rainbow packing.	1.30
8 lbs. 2-in. piston packing	7.04
15 oil cans.	4.50
5 grease cans.	1.00
15 gals. cylinder oil	6.75
1 5-lb. can cup grease	.50
1½ ft. Rainbow packing	.73
1 5-gal. oil can	.50
2 lbs. cup grease	.20
1 5-gal. can.	.50
3 lbs. piston packing	2.61
2 gals. boiler paint.	2.30
Total for engine supplies	$91.07

GRAVEL PIT MACHINERY

Article.	Cost.
6 10-in. wood pulley blocks	$ 9.00
1 gravel bin and screen	461.03
1 72-ft. mast.	104.51
1 Sauerman drag line, complete	
1 tank pump and hose	8.50
2 1-gal. oil cans	.35
1 5-gal. oil can	.50
3 tons coal.	13.50
2 lubricators.	9.34
1 ½-in. Glove valve, new	.92
1 injector.	12.00
12 1-in. nipples.	.60
1 box 12x½-in. bolts	.75
1 locomotive window.	
1 4-ft. bar.	.55
1 rake.	.75
3 lbs. packing.	1.95
1 Borsch plow.	24.00
3 slip scrapers.	12.00
1 Penberthy injector.	4.05
1 Austin crusher complete	
2 perforated screens.	22.00
16 ft. 1-in. chain.	5.71
Total for gravel pit machinery	$4,050.01

TAR OUTFIT.

7 wire street brooms.	$ 5.25
4 street broom handles.	.60
4 tar-spreadng cans.	6.00
Total for tar outfit	$11.85

SMALL TOOLS.

Article.	Cost.
1 4-in. open-end wrench.	$ 0.40
3 6-in. open-end wrenches.	1.65
2 8-in. open-end wrenches.	1.40
1 10-in. open-end wrench	1.80
1 12-in. open-end wrench.	1.00
2 14-in. open-end wrenches.	2.25
1 16-in. open-end wrench.	1.20
1 18-in. open-end wrench.	1.25
8 large open-end wrenches.	12.00
25 medium open-end wrenches.	18.75
1 14-in. pipe wrench.	1.00
1 16-in. pipe wrench.	1.12
5 18-in. pipe wrenches.	6.65
1 20-in. pipe wrench.	1.50
2 22-in. pipe wrenches.	3.60
6 24-in. pipe wrenches.	10.80
1 30-in. pipe wrench.	3.20
1 36-in. pipe wrench.	3.60
1 ¾-in. socket wrench.	.29
4 assorted socket wrenches	1.60
16 assorted socket wrenches	3.20
3 24-in. spanner wrenches.	1.35
1 12-in. Keystone wrench.	1.20
1 12-in. combination pipe and monkey	1.65
10 10-in. monkeywrenches.	11.00
2 14-in. monkeywrenches.	2.50
2 18-in. monkeywrenches.	3.20
1 8-in. monkeywrench.	.45
7 water pails.	22.90
1 tin dipper.	.05
1 granite dipper.	.30
1 washbasin.	.25
61 lanterns.	36.05
3 globes.	.75
18 claw hammers.	10.80
1 12-lb. sledge.	1.08
1 4-lb. sledge.	.80
1 5-lb. sledge.	.85
1 6-lb. sledge.	.90
1 8-lb. sledge.	1.00
1 10-lb. sledge.	4.40
5 stone hammers.	5.75
6 machine hammers.	3.30
1 machine hammer handle.	.15
650 ft. ⅜-in. wire cable	27.10
113 ft. ⅝-in. wire cable	6.94
100 ft. ½-in. wire cable	4.83
12 pc. ⅜-in. wire cable, 2 clamps	2.40
5 trowels.	6.25
1 edger.	.60
6 small squirt cans.	2.46
6 barrel faucets.	4.20
1 ball binder twine.	.50
3 50-ft. tapelines.	1.50
6 chalklines	.90
1 blower and forge (blacksmith)	12.00
1 anvil	8.63
1 vise.	5.60
2 bit stocks.	1.50
2 pair tongs.	.60
25 lbs ½x2-in. bolts	1.25
1 grindstone.	4.00
1 set stocks and dies	11.00
1 brace.	.75
11 bit.	4.25
1 post drill, etc.	10.17
2 hand augers.	1.50
3 post augers.	2.25
1 post hole drill	1.15
1 1-in. drill 18 ins. long	.50
1 1-in. drill 10 ins. long	.30
128 No. 2 shovels (1 broken)	89.60
6 No. 2 shovels	4.50
17 scoop shovels (2 broken)	9.92
9 long-handled shovels.	7.20
9 round-point No. 2 shovels	6.57
1 hatchet.	.95
1 hand axe.	1.00
5 axes.	5.85
12 grub axes.	9.00
48 picks.	32.85
7 pick handles.	1.05
2 axle handles	.40
2 scythe handles	3.20
1 grub axe handle	.30
12 hand saws.	15.60
3 crosscut.	5.25
4 hacksaw blades.	.40
2 saw sets.	.85
1 saw vice	1.30
1 try square.	.50
10 squares.	8.50
5 levels.	6.00
6 ash hoes.	5.00
3 grub hoes.	2.40
6 clinker hooks.	4.50
19 cold chisels	5.18
6 cold chisels (blacksmith)	9.00
6 crowbars.	4.43
3 wrecking bars.	1.95
1 4-ft. bar.	1.50
2 planes.	2.70
1 14-in. wooden plane.	1.75
1 20-in. wooden plane.	1.95
3 drawknives.	3.90
1 plumb bob.	.15
1 screwdriver.	.20
1 pair scissors.	.25
1 pair tin snipes.	1.25
2 putty knives.	.50
1 branding iron.	20.00
9 pair pliers.	5.80
12 12-in. flat files.	1.80
1 gauge glass cutter	1.20
1 jackscrew.	3.28
1 8-ton ratchet jack.	12.00
2 emery bricks.	1.00
1½ kegs nails.	4.50
12 large tool boxes.	96.00
1 small tool box	2.00
1 master mechanics' outfit	41.84
1 paint brush.	.20
1 flue roller.	4.00
Total for small tools	$742.00

DRAFTING ROOM.

Article.	Cost.
1 wood straight edge.	$ 0.75
1 steel straight edge.	6.40
½ roll profile paper.	3.13
¾ roll cross section paper	7.50
½ roll drawing paper.	3.25
1 roll tracing paper.	2.40
1 roll tracing cloth.	9.00
1 small ruling pen.	1.35
1 large ruling pen.	1.80
1 double ruling pen.	3.80
1 bow pen (ink), 1 bow pen (pencil), and bow pen dividers.	3.98
1 paper cutter.	.40
1 planimeter.	33.00
1 dotting machine.	4.50
1 stadia slide rule.	8.00
Total for drafting room	$89.26

SURVEYING APPARATUS.

Article.	Cost.
1 transit and tripod	$225.00
1 level and tripod	125.00
1 level rod.	12.00
1 8-ft. range pole	2.25
2 5-ft. range poles	4.00
1 50-ft. tape and case (metallic)	2.60
1 100-ft. tape (metallic)	4.20
1 set of pins (11)	1.25
2 100-ft. steel chains	8.00
2 small levels, tripod and box	27.00
1 tripod.	2.00
2 hand levels.	6.00
1 axe.	.75
1 reading glass.	.45
Total for surveying apparatus	$420.50

LABORATORY.

Article.	Cost.
1 analytical balance.	$110.00
1 set weights.	15.00
1 balance.	5.15
1 stop watch.	6.00
1 New York laboratory pentrometer	60.00
1 Ell. oil tester.	6.50
1 New York float test apparatus	8.00
1 gas reg.	4.00
1 steam bath.	3.00
1 8x10 drying oven.	5.00
2 iron stands, 20-in	.45
2 iron stands.	1.00
1 set 4 rings	.45
3 condenser clamps.	.90
3 Burrette clamps.	.60
6 small clamps.	.80
2 clamps.	.70
6 pieces wire gauze.	.33
1 pair tongs.	.65
1 filter pump.	1.00
1 filter pump conn.	.30
3 thermometers, 110 degrees C	2.00
3 thermometers, 250 degrees C.	2.70
2 thermometers, 400 degrees C.	3.30
4 hydrometers.	1.23
1 hydro jar.	.18
1 Hubbard picnometer.	.80
1 crystal dish.	.10
1 crystal dish	.08
12 retorts, 500 C. C	2.70
2 retorts, 1,000 C. C	.70
6 retorts 250 C. C	.90
2 wash bottles, 16-oz.	4.20
2 wash bottles, 16-oz.	1.50
1 wash bottle, 32-oz., flexible tube	.60
3 cylinders, 100 cu. c.	.90
3 cylinders, 25 cu. c.	.54
1 dessicator, 6-in., with porcelain plate	1.15
3 filter tubes.	.54
12 ft. 5-16-in. rubber tube	1.95
2 ft. 1¼-in. rubber tube	.48
5 lbs. heavy asbestos paper.	.75
1 asphalt flowt plate and mold	5.00
1 plat. triangle 5 cm. sides	19.61
1 Gooch plat. crucible, 25 gm.	39.41
2 pa. filter papers, 9 cm	1.60
1 pa. filter papers, 9 cm	.80
3 clamps.	1.14
1 clamp.	.14
1 small clamp.	.13
2 laboratory aprons.	1.60
2 alcohol lamps.	.48
2 pipettes, 100 C. C	.54
1 pair pliers, 5-in	.40
1 wood support.	1.25
12 watch glasses, 2½-in	.38
6 watch glasses, 4½-in	.45
6 por. crucibles, No. 0	.96
4 Royal Berlin crucibles, No. 2	1.15

4 Gootch crucibles, 35 mm.	1.20
2 iron sand baths, 6-in.	.32
12 clay triangles, assorted	.70
1 Soxhlet extractor, 210 C. C.	2.20
1 condenser, 12-in.	.60
1 hydrometer, naphtha, 90 degrees	.40
2 separatory funnels, 125 cc.	1.35
1 Westphal balance.	25.00
1 foot blower.	5.50
12 rubber policeman.	.70
1 set weights.	2.00
1 doz. test tube brushes	.60
2 pair bal. watch glasses, 3-in.	1.10
6 Burette brushes.	.45
2 lbs. asbestos washed in HNO_3	3.00
Difference between express and freight	6.35
1 plat. crucible, 25 gm., with cover	43.22
1 pair rubber gloves	1.34
1 iron mortar.	1.00
1 wedgewood mortar.	.60
1 horn scoop.	.09
1 aluminum scoop.	.09
1 test tube rack	.35
¼ gross 6-in. test tubes	.75
12 test tubes, 7-in.	.30
5 gross 2-6 corks	1.30
5 gross 6-10 corks	2.65
¼ lb. mercury chloride	.34
½ gal. cottonseed oil.	.48
6 12-oz. tall beakers	.90
5 32-oz. tall beakers	1.57
1 sign plate.	.50
24 rubber stopples.	.33
1 lb. glass tubing, assorted	.40
1 lb. glass rods	.50
2 10-in. blowpipes.	.50
1 doz. 2½ glass funnels	1.56
6 4½-in. evap. dishes.	2.25
12 8-oz. round glass bottles	1.31
12 16-oz. round glass bottles	1.50
24 16-oz. salt mouth bottles	3.76
24 32-oz. salt mouth bottles	4.88
12 8-oz. salt mouth bottles	1.69
1 copper water bath	1.50
1 tripod, 2-ring.	.35
2 tripods, 1-ring.	.60
6 Bunsen burners.	2.10
1 3-ring tripod.	.50
3 spattles, 4-6-8.	.95
1 2-in. bristle brush, black	.15
1 set cork bores.	.90
6 nests beakers, 1-6	4.20
2 No. 4 tall beakers	.30
9 Earl & Myer flasks, 100 cc.	1.50
2 Earl & Myer flasks, 250 cc.	.40
2 Earl & Myer flasks, 500 cc.	.60
10 lbs. carbon disulphide	1.80
5 lbs. carbon tetrachloride.	1.40
5 lbs. Squibs ether.	3.60
5 lbs. Squibs alcohol.	6.08
5 lbs. carbonate ammonia.	.70
5 lbs. metallic mercury.	4.00
5 lbs. calcium chloride.	1.40
9 lbs. C. P. sulphuric acid	1.33
6 500-cc. heavy flasks, side necks	2.03
6 retorts with tub. and stopper	2.25
5 gals. 86-deg. B. naphtha	1.75
3 each rubber stopples 00-8 inc.	1.39
1 gross 2 oz. tin boxes	1.80
36 3-oz. tin boxes	.60
Freight prepaid.	.75
1 cement testing machine	90.00
1 scale.	10.00
1 set Gilmore needles.	6.00
1 scale and weights.	4.50
1 set cement sieves with lid and bottom	10.00
1 set sand sieves with lid and bottom	7.00
2 trowels.	1.50
1 sampler.	7.50
1 sand glass.	.75
1 La Chatelier spec. grav. apparatus	4.00
100 lbs. a. s. s. e. sand	4.00
24 cans with covers.	2.00
1 Vicat needle.	34.65
4 4-gang moulds.	40.00
Freight.	2.60
1 4-gal. jar.	.40
3 blasting spoons.	.38
2 pans.	.40
4 pans.	3.60
6 tincups.	.45
1 2-qt. measure.	.20
2 brushes.	.12
1 glass cutter.	.35
1 ¾-in. cold chisel	.30
1 6-in. level.	1.00
1 N. Y. testing lab. drying oven	25.00
1 hydrometer 1.0 to 1.2	.42
Express.	1.60
1 Engler viscosometer	32.50
Express.	1.20
Water and bottle	1.50
1 pair tin snips.	.84
1 fish broiler with brass strainer	4.50
¾ lb. copper	.25
1 ¼-in. mesh screen.	.36
1 ¾-in. mesh screen.	.48
1 ½-in. mesh screen	.72
4 pieces galvanized iron.	.20
1 corkscrew.	.25
1 camera.	56.50
Camera supplies.	2.00
Total for laboratory	$861.88

CAMP

1 icebox.	$ 32.50
1 range and outfit.	107.27
2 flour scoops	.55
1 flour sifter	.20
6 granite washbasins.	2.40
1 roling pin.	.20
1 soup ladle	.25
1 16-in. knife steel	1.40
1 meat cleaver	.75
2 sad irons	1.08
3 lamp chimneys.	.30
1 meat saw.	1.15
3 drop cake pans.	.75
5 small oil lamps and holders	1.00
1 box wood screws.	.35
4 granite meat platters.	2.60
6 sugarbowls.	3.00
12 salt and pepper shakers	2.70
34 cups.	4.25
47 saucers.	5.48
3 roasting pans.	11.25
1 large flour sifter.	.35
1 chopping bowl.	.75
3 large dishpans.	3.75
1 small dish pan.	.24
1 wash boiler.	1.50
4 basting spoons.	.60
6 soup ladles.	.75
4 soup skimmers.	.74
1 meat chopper.	3.85
1 vegetable chopper.	2.00
1 carving knife.	.60
1 small creamer.	.50
2 potato knives.	.20
1 ½-gal. jug.	.08
1 1-gal. jug.	.10
1 egg beater.	.65
1 funnel.	.45
1 potato masher.	.65
6 tin kettle covers.	.50
10 pie plates.	1.50
1 5-gal. coffee kettle.	4.00
1 scrubbing brush	.20
1 pickle jar.	.20
1 3-gal. jug.	.18
1 2-gal. jug.	.12
1 galvanized water pail	.50
1 teakettle.	1.25
2 granite water pails.	1.50
1 granite water pail.	.75
1 granite coffee pot.	.50
2 water dippers.	.50
2 stewing pans.	6.50
1 pickle jar.	.20
1 soup strainer.	1.25
2 wire dish strainers.	1.20
30 tablespoons.	2.25
57 teaspoons.	5.09
70 knives and forks.	3.50
4 frying pans.	2.40
6 cooking kettles.	4.15
6 small pans.	.60
7 bread pans.	.70
3 pieces oilcloth.	2.20
1 large soup kettle.	1.35
1 nutmeg grater.	.03
1 2-qt. jar spices	1.00
4 lbs. ground ginger.	.76
13 sheets.	6.37
1 alarm clock.	1.00
11 pillow slips.	1.37
21 double roller towels	4.38
12 double roller towels	2.50
1 washboard.	.55
6 coffeepots.	6.00
9 granite cups.	1.15
14 granite plates.	3.50
7 granite 1-qt. creamers	1.40
1 galvanized pail.	.50
3 pieces 10-in. stovepipe	.60
1 dustpan.	.10
2 large vinegar barrels.	3.25
9 granite soup dishes.	1.35
2 cake pans.	.20
1 crockery bowl.	.65
4 tin pans.	1.00
8 stewing kettles.	4.00
1 potato kettle.	.50
4 tin covers.	.20
40 granite soup bowls.	6.00
12 granite soup bowls.	1.8
52 granite plates.	13.00
2 crockery saucedishes.	.70
5 syrup pitchers.	1.05
1 bread board.	2.50
1 sink.	13.50
1 washtub.	1.00
1 mop handle.	.45
Total for camp	$322.6

Detailed Study of Methods and Costs of Road Construction Undertaken by U. S. Office of Public Roads.—To determine so far as possible the points of excellence or defects in existing local methods of building and maintaining roads which will aid the state authorities in placing local road management on a systematized basis, the division of road economics of the United States office of public roads has begun a study of such work in 100 counties in the various states of the Union. It is understood that the highway department heads are co-operating cordially in this work, and it is expected that satisfactory results can be accomplished in this manner. These studies include: Methods of financing bond issues and road taxes; statute labor; prison labor; accounting and cost keeping; and the adaptability of various materials to local conditions. L. W. Page is director of this office and the study outlined is in charge of his assistant, J. E. Pennybacker, chief of the division of road economics.

The city of Atlanta, Ga., recently purchased a portable asphalt mixing plant for $13,500, and, it is claimed, will save $20,000 to $25,000 annually by its use.

Treatment of Wood Blocks in Berlin, Germany.

The size of blocks, method of treatment by zinc chloride and creosote oil and the rate of wear of wood blocks used in Berlin pavements are treated in a recent U. S. Consular Report.

The wood blocks mostly used are Swedish pine and, to a more limited extent, the Austrian hardwood varieties—tallow wood and blackbutt. The native pine and beech blocks are, however, also used in Germany, as are the American cypress and yellow and pitch pine. The wood used must be carefully selected, must be free from knots and cracks, and it is considered unwise to use in the same section of pavement woods from different countries, wood gathered at different altitudes, or wood of different species.

The dimensions of the paving blocks vary, the width ranging from 2.75 to 3.94 ins., the length from 7.09 to 11.81 ins., and the height from 4.72 to 7.09 ins. The so-called Paris standard dimensions are 2.95 by 8.86 by 5.9 ins.

To prevent decay the wood blocks are impregnated with a zinc chloride solution or with creosote oil. The zinc chloride solution contains 2 parts by weight of chloride of zinc, 3° Baume, and 30 parts by weight of water. The creosote oil should not contain less than 5 per cent of creosote and not more than 15 per cent of naphthalene and is distilled between 180° and 270° C. (336° and 518° F.). The specific gravity of the oil is 1.07.

The creosote oil impregnation is considered preferably, because it also serves to forestall the subsequent swelling of the blocks by dampness after they have been set and the resulting bulging of the pavement. To impregnate with creosote oil, the blocks are dried out for three to four hours in a tank or boiler at a temperature of 100° to 140° C. (212° to 284° F.), when they are transferred to another tank and allowed to remain for an hour in a vacuum of one-tenth to one-fifth of an atmosphere after which creosote oil that has previously been heated to approximately 50° C. (122° F.) is pumped in and forced into the wood under a pressure of six to eight atmospheres.

If the pavement is kept clean and if good materials have been used in construction, under ordinary conditions of traffic and of weather, the surface wears at the rate of about 0.2 in. annually. Wood block pavements on an average last 10 to 15 years, but in dark, damp places, not exposed to sunshine, the blocks will decay in half the time. The approximate cost of block pavement in Berlin is 14 marks per square meter ($2.79 a square yard).

Proportioning Gravel Concrete.

To the Editors:—In the article on "Proportioning Gravel Concrete: Quality and Cost Curves." There is a statement which should not be allowed to stand without some qualification. In the middle column of page 623 just below Fig. 5 is the following: "The amount of stone needed is 0.84 cu. yds. and 1 cu. yd. of pit run gravel contains 0.56 cu. yds. of stone, therefore, the amount to screen is $\frac{0.84}{0.56} = 1.5$ cu. yds."

This involves screening out the sand required in the concrete and remixing. The proper way is to determine the amount of bank gravel which contains 0.42 cu. yds. of sand, in this case 0.7 cu. yds., and screen the rest of the 1.5 cu. yds. or 0.8 cu. yds. This change reduces the cost of screening from 45 cts. to 24 cts. and reduces the cost of material for 1 cu. yd. of concrete the same amount. It thus requires the cost curves in figures 3, 4 and 5 to be remodeled as well as the curve of yards to screen in Fig. 2.

There are so many people who take the gospel of cost from your valuable paper that these statements should be corrected.

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452 Dominion St., Winnipeg, Canada.

Engineering and Contracting

Devoted to the Economics of Civil Engineering Design
and to Methods and Cost of Construction

Volume XLI. CHICAGO, ILL., JUNE 17, 1914. Number 24.

Engineering Critics.

In literature and in art we have critics who are recognized for their ability to pick out inaccuracies and defects. These critics certainly are not the most beloved of persons, yet they are needed and do much to insure better literature and better art. Their adverse criticism is feared, while their favorable criticism is much desired, and as a result writers and artists feel the necessity of putting forth their best efforts.

We now have a few public spirited engineers who are giving considerable attention to criticisms of some present tendencies in engineering practice. They realize, as do most of us, that adverse criticism, even though merited, is apt to be little appreciated by practicing engineers and that such criticism does not tend to increase their personal popularity. That these men have the courage to voice their convictions is to be commended. This kind of work requires true courage and unselfishness, and the only men who can accomplish much in it are those who are recognized as having a sound knowledge of engineering principles supplemented by long and practical experience. There is a tendency on the part of some engineers to feel that any adverse criticism of their work is a personal insult, as they do not seem to realize that their success or failure must be borne largely by the engineering profession.

In the past many engineers were loath to permit the publication of any part of their plans or any descriptions of methods used by them. This was due to their desire to avoid publicity, to the feeling that their work was their personal property, or to the fear of adverse criticism of their designs. Engineers and contracting firms are just beginning to appreciate fully that such narrow views constitute a mistaken policy and often serve to retard their progress. We know of many successful firms who are entirely willing to make public intimate details of methods and systems used by them, and thus be judged by their work. On the other hand, the structural engineer for a building which recently failed during construction informed us that he had copyrighted the plans of this building and would prosecute anyone who published any part of them. Most engineers, however, will grant that they have little right to success unless they are willing to stand on the merits of their designs.

By the term "critic" we mean one who is skilled in judging of the merits of engineering work; and we believe there are too few of such men who are willing to give the engineering profession the benefit of their opinions. New engineering data are constantly being published and new formulas suggested, and it is to be regretted that such information does not bring forth more discussion from experienced engineers. The tendency is to accept experimental data and to make them the basis of engineering practice without careful analysis. A number of our present rules and so-called working formulas possess little merit. They may, perhaps, save a little time, but such a saving is usually made at the expense of efficient designing. We do not advise every engineer to voice his criticism of engineering practice and its problems, but competent, public spirited engineering critics are needed.

Concrete.

The word concrete is derived from the Latin word *concrescere*, meaning to grow together. Hence any material formed by coalition of separate particles into one mass is concrete. This incursion into the etymology of a word very familiar to engineers is prompted by the letter quoted below from a valued correspondent:

> I read a great deal about Bituminous concrete and Asphaltic concrete. The phrase Portland cement concrete is also used. What is concrete? When we speak of concrete sidewalks, reinforced concrete or concrete construction, it is generally understood what is meant. Is the tendency to speak of Asphaltic concrete, Bituminous concrete, etc., to be encouraged? What is your view? The situation is rather a confusing one it seems to me and one with which some committee on nomenclature ought to deal.

The confusion of our correspondent seems to result from his conscious or unconscious assumption that concrete used as a name for a material of construction has by custom or usage become restricted to a composite material of which hydraulic cement alone is the binding element. This is a doubtful assumption. But even if it were wholly justified we should not ascape from the necessity of modifying phrases in speaking of concrete. In the United States concrete is made using slag cement, sand cement, tufa cement and natural rock cement as well as Portland cement. In Europe the list of hydraulic cements employed regularly in making concrete includes also trass cement, hydraulic lime, puzzolan cements and various other combinations which set hard when mixed with water. On this showing solely it is absolutely impossible by the word concrete alone to afford precise understanding; there must be used to modify the word concrete the class name of the cement of which the concrete is made. To say that the Arrowrock dam will be a concrete dam is not precise. It is even misleading. Since Portland cement is in America almost universally used as compared with other cements one assumes when it is said that a dam is concrete that it is a Portland cement concrete dam. But the Arrowrock dam is not being built of Portland cement concrete—a sand cement concrete is being used. So also large portions of the Los Angeles aqueduct were constructed of tufa cement concrete, a material, it is said, with which the Association of American Portland Cement Manufacturers are not anxious to have Portland cement concrete confused.

There is nothing in the root meaning of the word concrete which precludes its application to a composite material of which bitumen is the binding element. In fact the makers of dictionaries universally define concrete as a mixture of gravel or pebbles or broken stone with cement or *with tar*. Such mixtures have long been used by engineers and the material bituminous concrete is new only in the sense that its use in large volumes for road construction is somewhat recent. So long as care is taken to modify by the word bituminous the word concrete, no confusion of meaning is likely to occur, and there is little chance that any road builder except through pure oversight will designate a bituminous concrete road as a concrete road. There is far greater likelihood of confusion in speaking of hydraulic cement concrete structures because the speaker fails to specify the *class* of hydraulic cement employed.

Acceptance Clauses in Road Specifications.

It has often been said that contractors as a class base their bidding prices for road work more on a knowledge of the personality of the engineer in charge of it than on the specifications. If this is true—and it undoubtedly is true in a measure—there is a clause which occasionally crops out in road specifications which changes the ordinary risk involved in bidding to a wild gamble.

We quote from the specifications of a county engaged in extensive highway improvements:

> The said progress estimates (Editor's Note: Estimates for monthly payments) shall not be conclusive upon the county that the work covered thereby has been done according to the contract, but the final acceptance of said work shall be by the Board of Supervisors.
>
> At the expiration of thirty-five (35) days after the final acceptance of said work, the contractor shall be paid the balance due under the contract after deducting therefrom any demurrage charges or other amounts which the county may be lawfully entitled to retain which have not been previously deducted.
>
> The words "Highway Commission" and "Board of Supervisors" shall mean respectively the Highway Commission and Board of Supervisors of said county.
>
> The word "Engineer" shall mean the chief engineer of said Highway Commission acting either directly or through properly authorized agents, such agents acting within the scope of the particular duties entrusted to them.

After completing the work "to the satisfaction of the engineer" the contractor must submit to this. Suppose the specifications for a railroad contract should contain the same clause with reference to the board of directors, with the board of directors separately defined in the specifications. Or suppose a city paving specification substituted board of aldermen therefor. In effect this clause not only multiplies the number of men upon whom final acceptance depends, several of whom may be totally unfamiliar with the details of the work and who may not have seen it until completed, but it deprives the engineer of all right to defend the work which he has supervised.

We do not care to comment further—it is unnecessary. Such clauses may be interpreted fairly, or they may not. To say the least, they are loosely drawn and vicious and their use cannot be too strongly condemned. We believe this clause has been interpreted fairly in this county, but we must confess that according to its terms the contractor's head appears to be in a noose.

As a contrast to this type of acceptance clause we quote from recent specifications of the State Highway Commission of Maine:

> The engineer shall, as soon as practicable, after the completion of this contract, make a final estimate of the amount of work done thereunder and the value of such work, and the Commission shall at the expiration of thirty days from and after such final estimate is so made and is approved by the Commission, pay the entire sum so found to be due hereunder, after deducting therefrom all previous payments and all amounts to be kept and all amounts to be retained under the provisions of the contract. All prior, partial estimates and payments shall be subject to correction in the final estimate and payment.

The Maine acceptance clause states specifically that the engineer shall act as the agent of the commission throughout the work, not merely on part of it. Any action of the commission must be through the proper channel, namely, the engineer it has employed.

We believe the engineer should have authority in matters of dispute and reputable contractors do not object to this, but the elimination of the engineer in the final acceptance of work and the substitution of a board is unfair both to the contractor and the engineer.

Bridges

Design Features of the Piers for the North Side Point Bridge, Pittsburgh, Pa.

(Staff Article.)

II.

In our issue of March 25, 1914, we described and illustrated some design features of the three main 531-ft. truss spans of the North Side Point Bridge, Pittsburgh, and gave a general description of the structure. In this issue we shall consider the design of the three main piers of this structure.

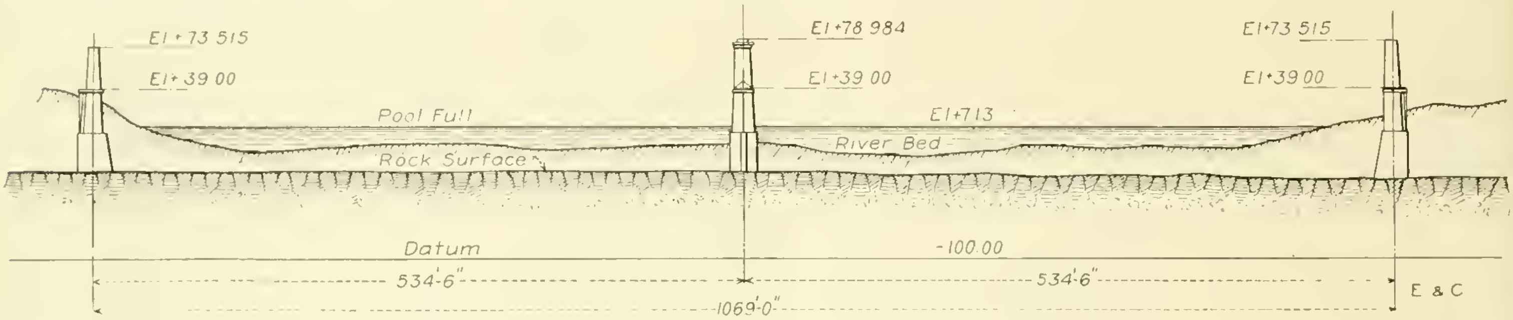


Fig. 1. Position of Piers for North Side Point Bridge and Profile on Center Line.

The three piers which carry the two main 531-ft. truss spans of the North Side Point Bridge rest on solid rock and are constructed of concrete, faced with stone masonry. An elevation of the piers showing the profiles at the site is given in Fig. 1. Pier No. 3, the south shore pier, carries a concrete arch span on the land side and one end of a 531-ft. steel truss span. Pier No. 4 is the river pier, and it carries the adjacent ends of the two main truss spans. Pier No. 5, the north shore pier, carries one end of a 531-ft. steel truss span and a concrete arch span, on the land side. From the base of the piers up to about the pool level the construction is concrete, encased in steel sheet piling. From the top of this concrete the material is stone facing and concrete filling, with steel grillage supports for the bearings of the steel spans. The three piers contain about 16,000 cu. yds. of concrete and stone masonry and about 365 tons of steel in the grillages. The contract price of these piers was $182,780.

CONCRETE AND STONE MASONRY.

The concrete in the bottom of the pier bases, up to elevation—6.5, was laid under water and consists of a 1:2:4 mix, while that above this elevation was laid in the dry and consists of 1:2½:5 mix. It was specified that the stone for the masonry should be of the best quality of Beaver, Baden or Somerset sandstone, or a stone of equal quality, the color-tone to be blue-gray. The concrete from the rock surface up to elevation—6.5 was deposited by means of tremies. It was specified that the tremies must be kept submerged in the flowing concrete to prevent water from entering them. After the concrete had been laid up to elevation—6.5, and had hardened, the water in the cofferdam was pumped down to the level of the concrete, and the remaining concrete was laid in the dry.

For the stone masonry it was specified that the facing should have not less than one header to every two stretchers, the stretchers to have a width not less than 1¼ times their depth and a length not less than twice their depth nor more than three times their depth, and the headers to have a length of at least 2½ times their depth and a width of at least 1¼ times their depth. It was required that the beds be dressed to vary not more than ¼ in. from a true plane, while the vertical joints were to be dressed to vary not more than ¼ in. from a vertical plane, for a distance of at least 8 ins. back from the face. The noses of the river pier, below the belt course, were finished with a 2-in. chisel draft on each side of the center line and were dressed, by fine pointing, for at least 1 ft. back of this. The part of the nose above the belt course was pean-hammer dressed on all exposed surfaces, while belt courses and the coping were also given the same finish for their entire depths and projections. The sub-copings were finished with a 2-in. chisel draft along the top and bottom edges and along the arrises, while all arrises, from sub-copings down to belt courses and from belt courses to the concrete bases, were given a 2-in. chisel draft.

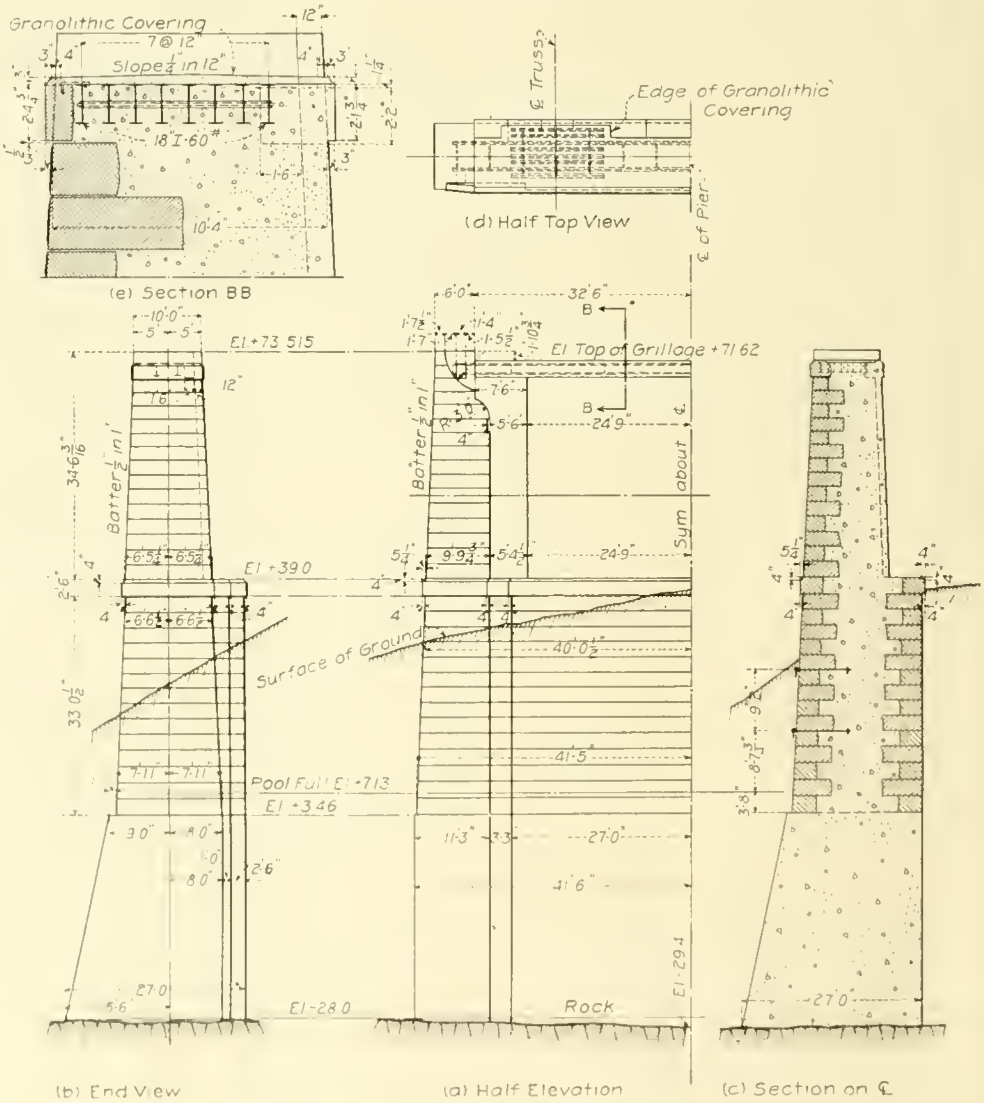


Fig. 2. Half Elevation, End View and Sections of North Shore Pier (No. 3) of North Side Point Bridge, Pittsburgh, Pa.

The mortar used in laying the stone masonry consisted of 1 part Portland cement and 2 parts sand, while that used for pointing consisted of equal parts of cement and sand.

DESIGN FEATURES.

South Shore Pier.—The south pier, No. 3, rests on solid rock, which lies about 66 ft. below the ground surface. This pier carries one end of a concrete arch span and an end of 531-ft. steel truss span, the bearings of the latter resting on an I-beam grillage embedded on the top of the pier. The bottom of the pier is 83 ft. wide by 27 ft. thick, the top is 77 ft. wide by 10 ft. thick, and its height is about 102 ft. Figure 2 (a) shows a half elevation of the south side of this pier and gives its principal dimensions; Fig. 2 (b) shows an end view; Fig. 2 (c) shows a cross-section on the center line of the pier; and Figs. 2 (d) and (e) show details at the top of the pier. By referring to Fig. 2 (c) it will be seen that this pier consists of concrete from the base up to a plane 3 ft. 8 ins. below the water level. Above this level the pier is faced with sandstone, except where the arch span joins it. Figure 2 (e) shows the granolithic covering which was applied after the superstructure was erected. This work was not done by the contractor for the substructure.

North Shore Pier.—The north shore pier, No. 5, is similar in construction to the one just described. This pier is also founded on solid rock, its height being about 108 ft. The principal dimensions are the same as those shown in Fig. 2. The bottom of this pier lies about 50 ft. below the ground surface.

River Pier.—The river pier, No. 4, is also faced with sandstone and has a concrete filling. The base of this pier is 83 ft. wide by 22 ft. thick, the top of the shaft is 53 ft. wide by 14 ft. thick, and its total height is about 107 ft. The pier is founded on solid rock, which lies about 26 ft. below the river bed. Figure 3 (a) shows a half elevation of the river pier and gives the general dimensions; Fig. 3 (b) shows an end view; Fig. 3 (c) is a section near the top of the pier shaft; and Fig. 3(d) is a top view of the pier.

On account of the heavy load carried by this pier, six deep built-up girders are embedded in its top, upon which rest the I-beam grillages, which in turn carry the bearings of the 531-spans. Figure 4 (a) (p. 686) is a section along the center line of the river pier. Fig 4 (b) is a half cross-section of the top portion of the pier; and Fig. 4 (c) is a half end view of the top of the pier. The latter drawing shows clearly the method used to distribute the loads over the pier.

PERSONNEL.

The North Side Point Bridge was designed under the direction of the Department of Public Works of Pittsburgh, Mr. John G. Armstrong, director; Mr. N. S. Sprague, superintendent; and Mr. Emil Swensson, consulting engineer.

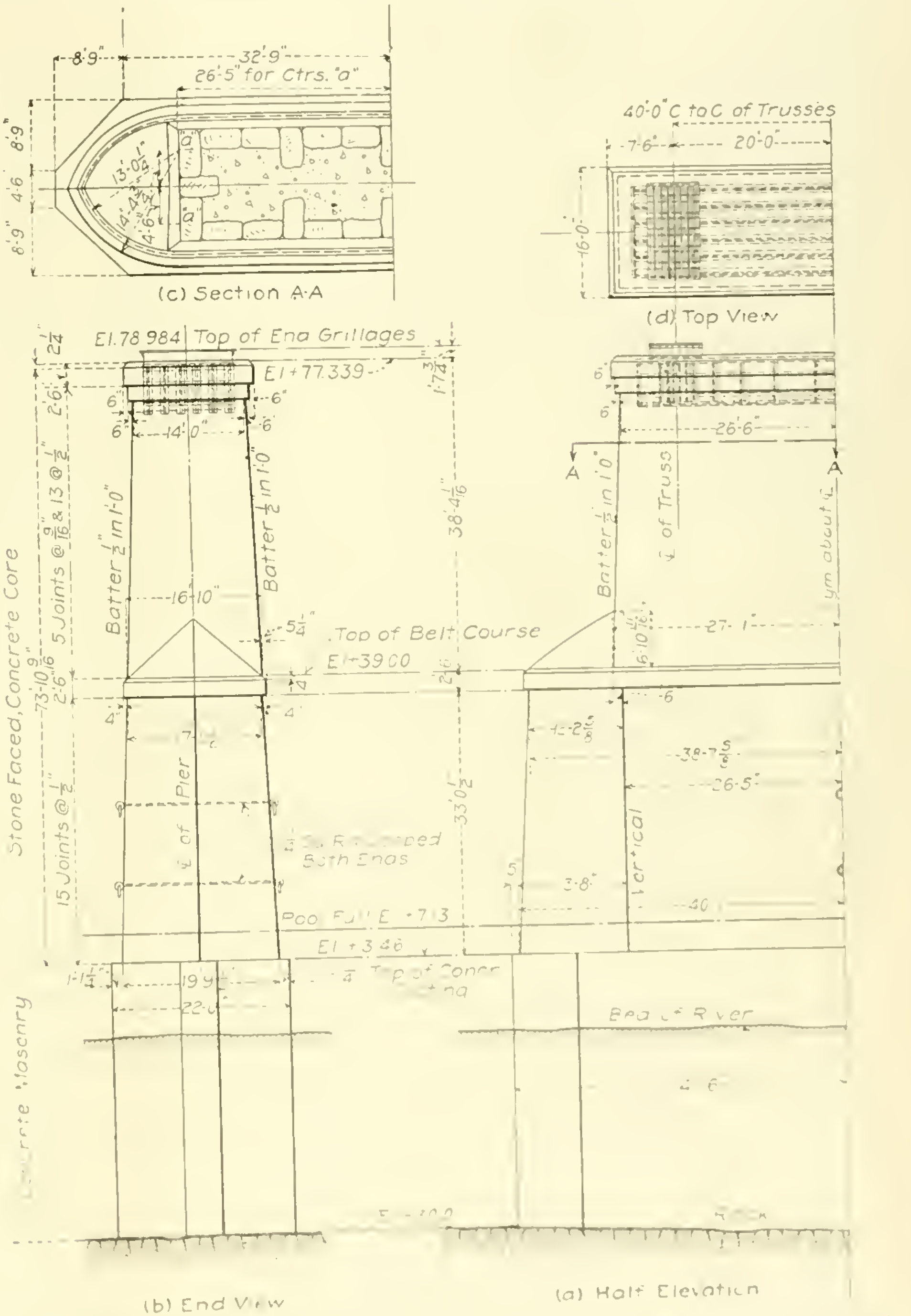


Fig. 3. Half Elevation, End View and Details of River Pier (No. 4) of North Side Point Bridge, Pittsburgh, Pa.

A Comparison of Carbon Steel and High-Alloy Steels for Bridges.

In designing bridges having exceptionally long spans it becomes necessary to use alloy steels of high elastic limit and ultimate strength to lessen materially the weight of the structure. Manufacturers of bridge metals have not progressed very far in their attempts to discover alloys of steel which possess high strength and also can be readily fabricated into the required sections. Nothing more satisfactory than nickel steel has yet been produced, and even this material is difficult to work. A number of important bridges have been built of nickel steel, although metal manufacturers are not yet willing to guarantee an elastic limit of 60,000 lbs. per square inch. The increased cost of nickel steel, and the fact that there is no market for rejected material, have made manufacturers unwilling to guarantee an elastic limit greater than 50,000 lbs. per square inch. The following data on "High-Alloy Steels for Bridges" were abstracted and rearranged from a paper by J. A. L. Waddell, in Proceedings, American Society of Civil Engineers, Vol. XL, p. 669, and from the discussion of this paper on p. 1613 of the Proceedings.

A condition which at present militates seriously against the use of nickel steel in bridge building is that the manufacturers ask for it an additional price of about 2 cts. per pound, as compared with ordinary carbon steel, although but little more than one-half that would be a sufficient excess price. As the future development of America will necessitate the construction of many long-span bridges it is almost a necessity that there be found an alloy of steel of great strength, high elastic limit, workable under all necessary manipulations in the shops, and of moderate cost. It is realized that the discovery of such an alloy will require much study and exhaustive experiments, but the saving in cost of a single large bridge might easily exceed the entire expenditure for such experiments.

The basis of the following investigation is a mass of diagrammed and tabulated data on the weights of metal in simple spans and cantilever bridges of carbon steel, up to a limit of 600 ft. spans for the former and 1,800 ft main openings for the latter, accumulated by the writer and his firm during the last 25 years, together with the weights of nickel-steel bridges and of mixed nickel-steel and carbon-steel bridges computed by the writer in the preparation of a previous paper on "Nickel-Steel for Bridges." As the weights of metal per linear foot in simple truss bridges were limited to lengths of 600 ft. in the former paper they have here been extended to 1,000 ft. by making actual calculations of stresses, sections, and weights of metal for several long spans, using the various kinds of steel assumed. The weights for bridges of carbon steel are based on the standard specifications given in the writer's "De Pontibus." They are quite accurate up to the limit of 1,000 ft for simple spans and 1,800 for the main openings of cantilever bridges. Figures 1 and 2 give the equivalent uniform live load per linear foot of single track assumed in computing the weights of trusses. The impact percentages were obtained from the writer's formula.

$$I = \frac{40{,}000}{L + 500}$$

, where I is the percentage of impact, and L is the loaded length, in feet, required to give the maximum stress. Figures 3 and 4 give a combination of the equivalent uniform live loads and the impact loads. The loads obtained from these curves added to the

TABLE I.—WEIGHTS OF METAL IN FLOOR SYSTEMS OF SIMPLE SPANS.

Metal mainly used in span.	Weight of metal per lin. ft. of span, lbs. For 350-ft. span.	For 600-ft. span.	For 1,000-ft. span.
Carbon steel.......	1,400	1,550	2,000
E = 50,000 lbs.....	1,150	1,300	1,750
E = 60,000 lbs.....	1,000	1,150	1,600
E = 70,000 lbs.....	900	1,050	1,500
E = 80,000 lbs.....	850	1,000	1,400
E = 90,000 lbs.....	800	950	1,300
E = 100,000 lbs.....	750	900	1,200

TABLE II.—WEIGHTS OF METAL IN LATERAL SYSTEMS OF SIMPLE SPANS

Metal mainly used in span.	Weight of metal per lin. ft. of span, lbs. For 350-ft. span.	For 600-ft. span.	For 1,000-ft. span.
Carbon steel	450	600	1,200
E = 50,000 lbs.....	450	600	1,150
E = 60,000 lbs.....	450	600	1,100
E = 70,000 lbs.....	450	600	1,050
E = 80,000 lbs.....	450	600	1,000
E = 90,000 lbs.....	450	600	950
E = 100,000 lbs.....	450	600	900

TABLE III.—WEIGHTS OF METAL ON PIERS FOR SIMPLE SPANS.

Metal mainly used in span.	Weight of metal per lin. ft. of span, lbs. For 350-ft. span.	For 1,000-ft. span.
Carbon steel	250	400
E = 50,000 lbs	200	300
E = 60,000 lbs	190	280
E = 70,000 lbs	180	260
E = 80,000 lbs	170	240
E = 90,000 lbs	160	220
E = 100,000 lbs	150	200

TABLE IV.—WEIGHTS OF METAL IN FLOOR SYSTEMS OF CANTILEVER BRIDGES.

Metal mainly used in span.	Weight of metal per lin. ft. of span, lbs. 600-ft. span.	1,200-ft. span.	1,800-ft. span.	2,400-ft. span.	3,000-ft. span.	3,600-ft. span.
Carbon steel	1,600	1,800	2,000	.....	.....	.....
E = 50,000 lbs	1,200	1,450	1,650	2,400	.....	.....
E = 60,000 lbs	1,000	1,200	1,400	2,100	.....	.....
E = 70,000 lbs	950	1,150	1,300	1,850	.....	.....
E = 80,000 lbs	900	1,050	1,200	1,700	2,200	.....
E = 90,000 lbs	850	950	1,100	1,600	2,000	.....
E = 100,000 lbs	800	900	1,000	1,500	1,900	2,400

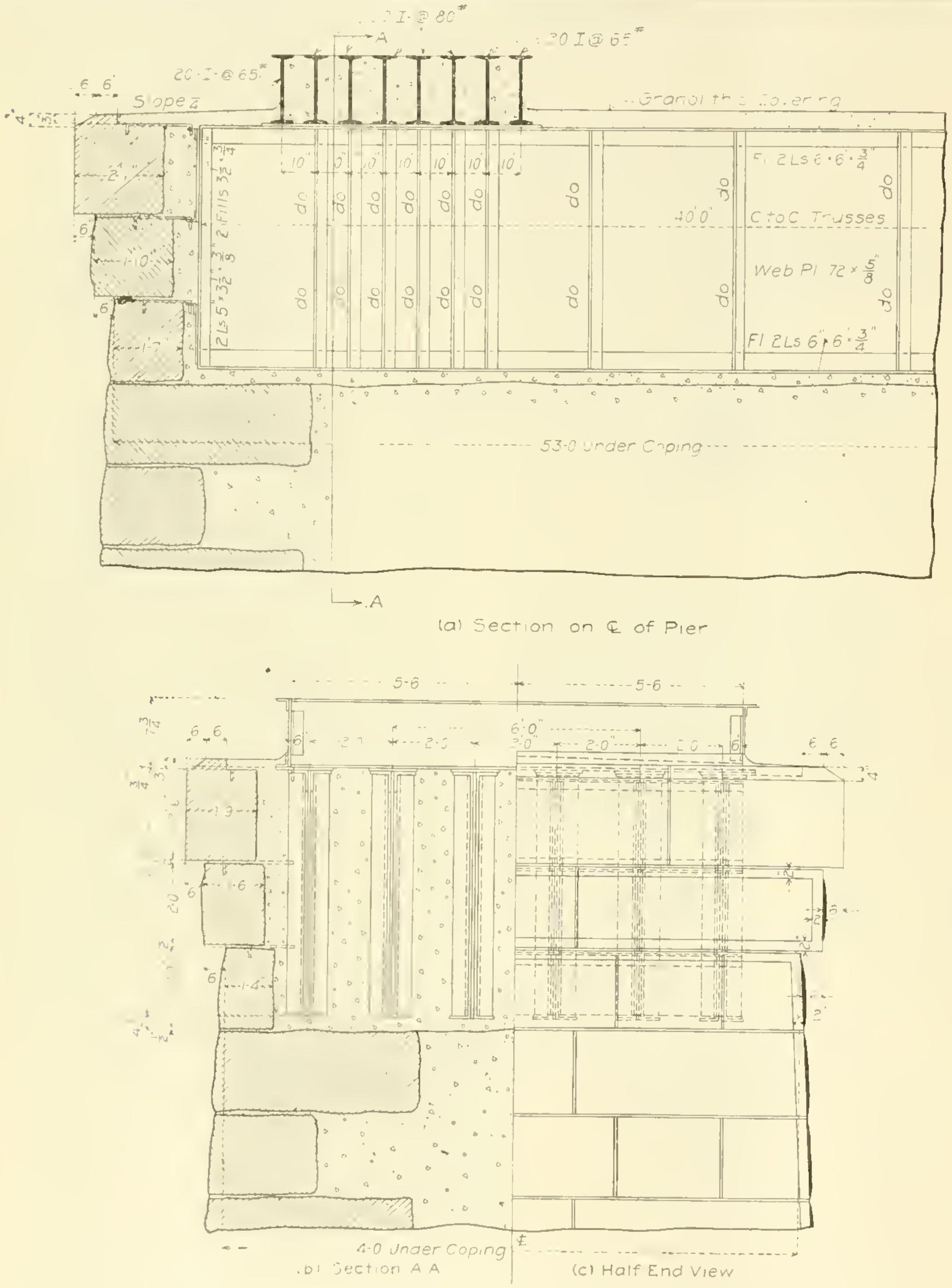


Fig. 4. Sections Showing Details of Top of River Pier and Grillage of North Side Point Bridge, Pittsburgh, Pa.

dead loads give the total loads per linear foot used for the bridges.

Tables I to VI, inclusive, give the approximate weights of metal, in pounds per linear foot of span, for the floor system, lateral system and on piers for simple and cantilever bridges, for various spans and elastic limits, E. From these tables and the diagrams shown in Figs. 5 and 6 (which will be explained later) there can be determined the weights for the trusses.

(The author develops reduction formulas for determining the weights of floor systems, lateral systems and on piers of alloy-steel bridges from the known weights of carbon-steel bridges, but we have omitted these in our abstract.)

The weight in pounds per linear foot of the trusses of simple carbon steel spans may be expressed by the formula,

$$T = K + T_1 + C_c + C_w,$$

where K is the part of the total truss weight per linear foot which is independent of the quality of the metal and of the stresses; T_1 is the weight of the main portions of the tension members and of their details which are directly affected by the stresses; C_c is the weight of the main portions of the compression chords and inclined end posts and their details which are directly affected by the stresses; and C_w is that of the main portions of the compression web members which are directly affected by the stresses. From experience in designing large bridges it may be stated that, as an average, $K = 0.2\ T$; $T_1 = 0.3\ T$; $C_c = 0.3\ T$; and $C_w = 0.2\ T$.

It is well known that in trusses with parallel chords and of economic depths the weight of the chords is equal to the weight of the web; but in trusses with polygonal chords, having center depths less than the theoretically economic ones, as do those of all long-span bridges, the weight of the chords is much greater than that of the web. As a general average for long spans the ratio of weight of chords to that of webs is about 6 to 4.

Figure 5 gives the total weight of metal per linear foot of span in simple-truss bridges for "Class R" live load, for carbon steel and for alloy steels having various elastic limits. An inspection of these curves shows the great saving in weight of metal which may be obtained by using alloy steels instead of carbon steel. This difference is most apparent between the weights for alloy steel having an elastic limit of 50,000 lbs. (the nickel steel which the manufacturers are willing to furnish) and that having an elastic limit of 60,000 lbs. The gradual reduction in the saving of metal with the increase of elastic limit is strikingly noticeable; and the conclusion may be drawn that, unless the extremely high-alloy steels can be obtained with only a moderate increase in cost, there will be no economy in using them for simple-span bridges.

Figure 6 gives the average total weights of metal per linear foot of span for cantilever structures having main openings of various lengths. The live loads used are "Class R" and "Class S" for the floor systems and "Class U" for the trusses. The proportional dimensions of typical, through, cantilever bridges are as follows: a main span, l, having a suspended span of ⅜ l, two cantilever arms each of 5/16 l, and two anchor arms of the same length as the cantilever arms. Any reasonable variation from these proportions would not change materially the average weight of metal per linear foot of span given by the curves of Fig. 6. The superiority of alloy steels over carbon steel is just as clearly shown as it was in the curves for the simple spans, but the advantage of using very high steels is greater.

If it is assumed that a limit of 36,000 lbs. of metal per linear foot of span is as high as

TABLE V.—WEIGHTS OF METAL IN LATERAL SYSTEMS OF CANTILEVER BRIDGES.

Metal mainly used in span.	Weight of metal per lin. ft. of span, lbs. 600-ft. span.	1,200-ft. span.	1,800-ft. span.	2,400-ft. span.	3,000-ft. span.	3,600-ft. span.
Carbon steel	800	1,100	1,400	.....	.....	.....
E = 50,000 lbs	800	1,050	1,300	1,800	.....	.....
E = 60,000 lbs	800	1,000	1,200	1,600	.....	.....
E = 70,000 lbs	800	1,000	1,200	1,600	2,000	.....
E = 80,000 lbs	800	1,000	1,200	1,600	2,000	.....
E = 90,000 lbs	800	1,000	1,200	1,600	2,000	2,400
E = 100,000 lbs	800	1,000	1,200	1,600	2,000	2,400

TABLE VI.—WEIGHTS OF METAL ON PIERS FOR CANTILEVER BRIDGES.

Metal mainly used in span.	Weight of metal per lin. ft. of span, lbs. 600-ft. span.	1,200-ft. span.	1,800-ft. span.	2,400-ft. span.	3,000-ft. span.	3,600-ft. span.
Carbon steel	700	1,100	2,100	.....	.....	.....
E = 50,000 lbs	600	900	1,700	.....	.....	.....
E = 60,000 lbs	580	860	1,600	2,200	.....	.....
E = 70,000 lbs	560	820	1,500	2,100	3,500	.....
E = 80,000 lbs	540	780	1,400	2,000	3,200	.....
E = 90,000 lbs	520	740	1,300	1,900	2,900	3,900
E = 100,000 lbs	500	700	1,200	1,800	2,600	3,600

it is either economical or practicable to go in the construction of double-track cantilever bridges (and the curves show this to be a logical limit), the following limiting lengths of main openings will be approximately as follows:

Kind of steel.	Maximum length, in ft.
Carbon steel	2,030
Steel with 50,000 lbs. elastic limit	2,340
Steel with 60,000 lbs. elastic limit	2,590
Steel with 70,000 lbs. elastic limit	2,780
Steel with 80,000 lbs. elastic limit	2,910
Steel with 90,000 lbs. elastic limit	3,030
Steel with 100,000 lbs. elastic limit	3,140

The assumption of 36,000 lbs. of metal per linear foot of span as a maximum means that, for carbon steel, there would be required at this limit 4.35 lbs. of metal to support each pound of live load (exclusive of impact allowance); and that for the alloy steels of various elastic limits the corresponding values are 4.37, 4.39, 4.40, 4.41, and 4.42, respectively, the average of which is 4.4 lbs. From the appearance of the curves at their upper ends one may draw the conclusion that, in the case of very high-alloy steels, the limit of weight of metal per linear foot of span can legitimately be raised beyond the 36,000-lb. limit. The more nearly these curves approach the vertical the more uneconomical it would be to extend the limit beyond 36,000 lbs. per linear foot. It is plainly evident that there is no advantage in carrying the carbon-steel bridges beyond the limit of 2,000 ft. for the main opening, but it is otherwise for the 100,000-lb. elastic limit steel. Continuing the curve for the latter it is found that the weight would reach 46,000 lbs. per linear foot for a span of 3,400 ft.; and that the inclination from the vertical at that point is greater than that for the carbon-steel curve at its limit of 36,000 lbs. with a main opening of 2,030 ft. Perhaps therefore it would be more correct to assume the extreme economic limit of main opening to be 3,400 ft. or even 3,500 ft. For this last length the average weight of metal per linear foot of bridge shown by the 100,000 lb. elastic limit curve would be 52,000 lbs., which means that it would require 6.38 lbs. of metal to support each pound of live load, exclusive of the effect of impact. Although this is an excessive quantity, it is nevertheless conceivable that conditions might exist which would render it advisable to adopt this extreme limit of main opening, although at such a length a suspension bridge would undoubtedly be cheaper. If it is admitted (as is maintained by some bridge engineers) that the impact of the live load on the main members of long-span trusses is immaterial, the practical limit of length of the main opening will be somewhat increased. Moreover, such a contention is not far from correct, as the latest experiments on impact from live loads in bridges show that its effect on moderately long spans is much less than engineers in general have been assuming. It is unlikely that the impact ever reduces to zero, but for openings of 1,200 ft. or greater it is true that its amount is so small as to be negligible, in view of the fact that the live load stresses in the main truss members will never be quite as large as they are computed. The latter statement is true, because (a) the trains on two tracks never advance together so as to produce maximum web stresses; (b) such trains are not likely ever to cover entirely the bridge or even any individual part of it, except, perhaps, the central span; and (c) it is improbable that any load of cars—unless they are ore or coal cars—will ever be uniformly full or loaded to the assumed limit.

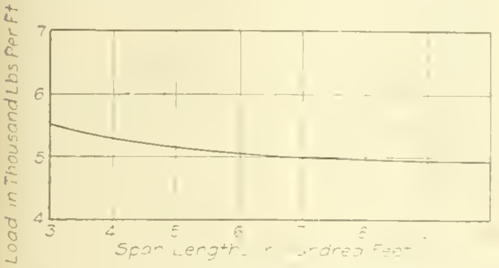


Fig. 1. Uniform Live Loads Equivalent to Class R Bridges.

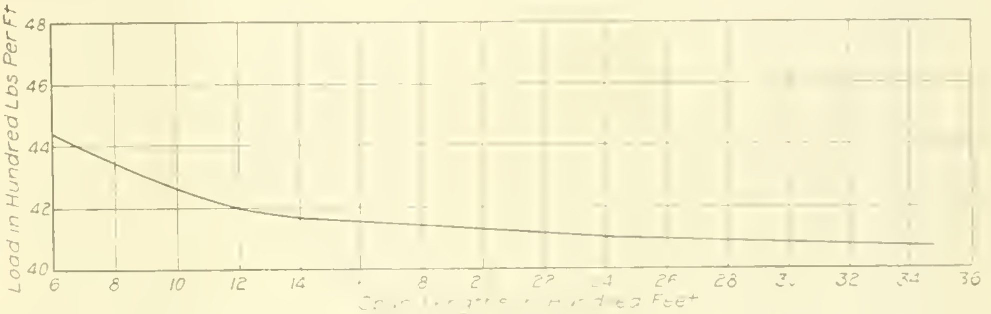


Fig. 2. Uniform Live Loads Equivalent to Class U Bridges.

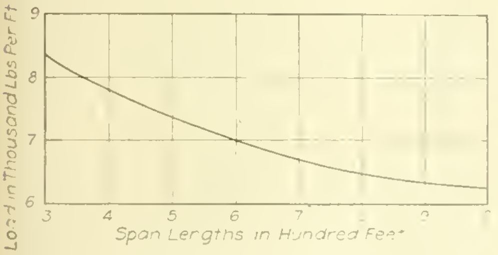


Fig. 3. Uniform Live Loads Equivalent to Class R Bridges Plus Impact.

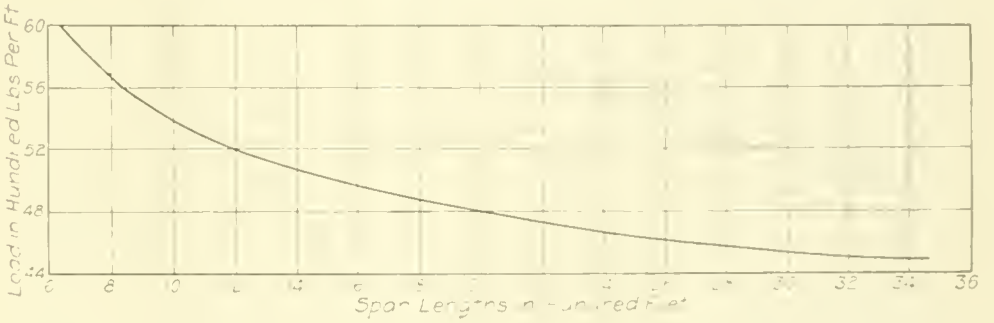


Fig. 4. Uniform Live Loads Equivalent to Class U Brioges Plus Impact.

Figure 7 shows the curves of weights for cantilever bridges of the same type and loading as those used in preparing the curves in Fig. 6, except that the impact on main members of trusses is assumed to be zero. These curves begin at main openings of 1,200 ft. and extend to the greatest practicable limiting lengths of such openings. A comparison of the curves of Figs. 6 and 7 shows that by neglecting impact on trusses there is an average saving of about 700 lbs. of metal per linear foot of span, for all spans and all kinds of steels. With a few exceptions this difference is comparatively uniform for all the curves, over their entire lengths.

Under the assumption that the limit of weight of metal is 36,000 lbs. per linear foot the greatest practicable span lengths have been increased on the average only about 20 ft. by neglecting impact on trusses. By comparing the extensions of the curves for steel having an elastic limit of 100,000 lbs it is found that, for an assumed limit of 52,000 lbs. of metal per linear foot of span, the extreme practicable length of main opening has been increased only 25 ft. These comparisons indicate that there is little gain, either in economy or increase of practicable limit of opening, in neglecting the effect of impact.

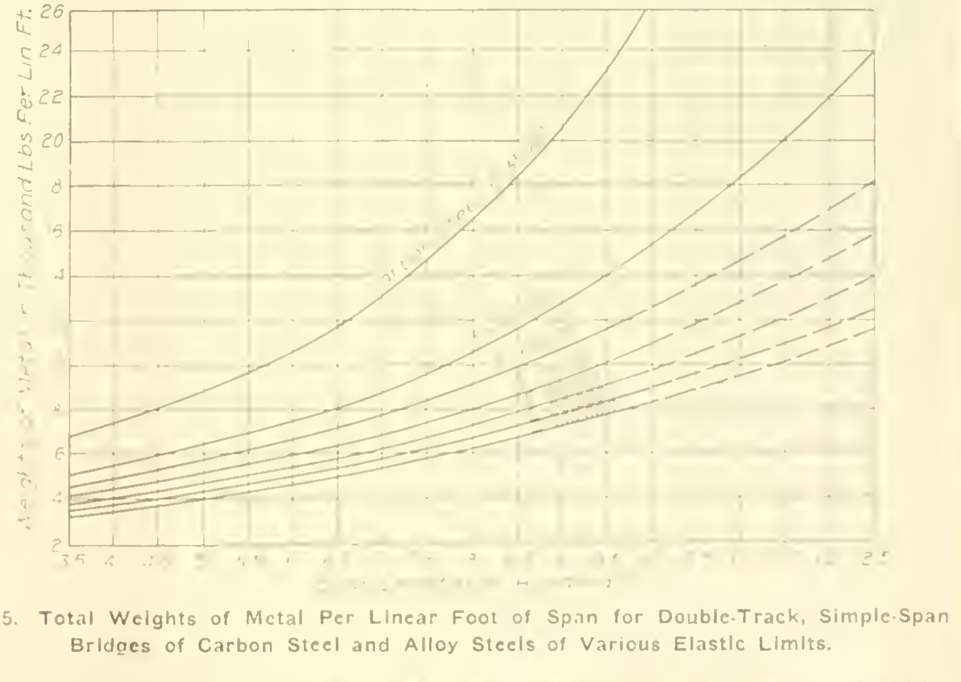


Fig. 5. Total Weights of Metal Per Linear Foot of Span for Double-Track, Simple-Span Bridges of Carbon Steel and Alloy Steels of Various Elastic Limits.

Figures 8 and 9 (p. 689) show the percentages of carbon steel in structures of mixed nickel and carbon steels. The curves are accurate for simple spans up to 600 ft. and for cantilever bridges with openings up to 1,800 ft., and beyond these limits they have been continued by deflections. These curves were prepared from diagrams of weights of metal in bridges of mixed nickel and carbon steels, and the diagrams are the result of careful, detailed computations of actual designs. The percentages are, of course, subject to great variation, because no two designers would agree exactly as to what minor parts of an alloy-steel bridge should be made of carbon steel.

(Considerable data on the relative cost of carbon-steel and alloy-steel bridges were given in the original paper, which we have omitted in our abstract, as the question of cost is an uncertain one. The writer states that he feels truly confident in regard to the correctness of the information contained in this paper.)

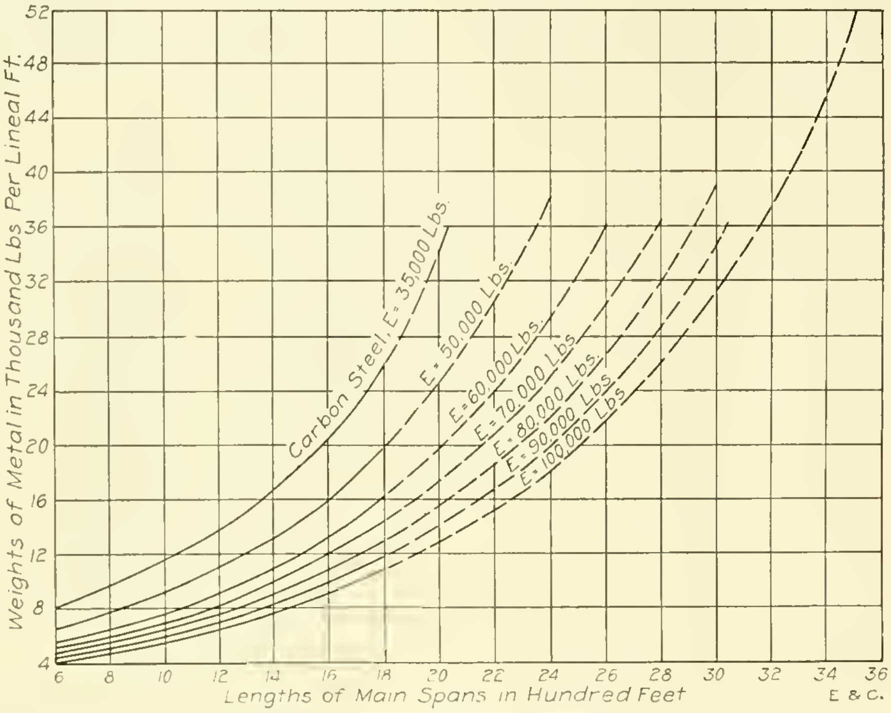


Fig. 6. Total Weights of Metal Per Linear Foot of Span for Double-Track, Cantilever Bridges of Carbon Steel and Alloy Steels of Various Elastic Limits.

Discussions.

The following abstracts from the discussions of the foregoing paper contain pertinent data:

DISCUSSION BY HENRY W. HODGE.

The length of bridge spans in general use has been increasing steadily, and we have reached limits where the dead weight of the structure has become the largest portion of its carrying capacity, so that some method of keeping down the weight is a necessity for the construction of the great spans now contemplated. The only way to reduce the dead load materially is by the use of metal of higher carrying capacity than our present materials, and an important step has been made in this direction by the use of nickel steel, which has 50 per cent greater carrying capacity than the carbon steel in general use.

The trusses of the three 668-ft. spans of the St. Louis Municipal Bridge were designed for nickel steel throughout, except certain minor sub-members. Nickel-steel eye-bars and carbon-steel compression members were also used, the floor system and bracing being of carbon steel in each case.

The weights of each span were:

With complete nickel-steel trusses, lbs..	9,200,000
With nickel-steel bars, and the rest of carbon steel, lbs.	10,900,000

As the dead load of railways, tracks, etc., was 5,500 lbs. per linear foot, the total average dead load was

With nickel-steel trusses, lbs. per linear foot	19,300
With nickel-steel bars, and the rest of carbon steel, lbs. per linear. foot	21,800

Thus, the nickel-steel compression members in the trusses made a difference in weight of 2,500 lbs. per linear foot, or 13 per cent.

The average live load on the two decks was 16,600 lbs. per linear foot, thus the use of nickel-steel compression members made a saving of 7 per cent in the total load on the structure.

The average unit prices for the two classes of material in this structure, erected in place, were:

Nickel steel, per pound	5.6 cts.
Carbon steel, per pound	3.95 cts.

The difference in cost of the two materials was 1.65 cts. per pound or 42 per cent of the price of the carbon steel; but the elastic limit required for the nickel steel was 50 per cent higher than for the carbon steel, so that the nickel steel was the cheaper, considering the strength.

The nickel steel had 3¼ per cent of nickel, but manufacturers are now commercially producing, at a very much reduced price, an alloy steel with not more than 1½ per cent of nickel, together with small percentages of chromium and vanadium, which has all the properties of this steel, so that there is at present a readily obtainable material, which is 50 per cent stronger than the carbon steel in general use, at a comparatively small increase in cost.

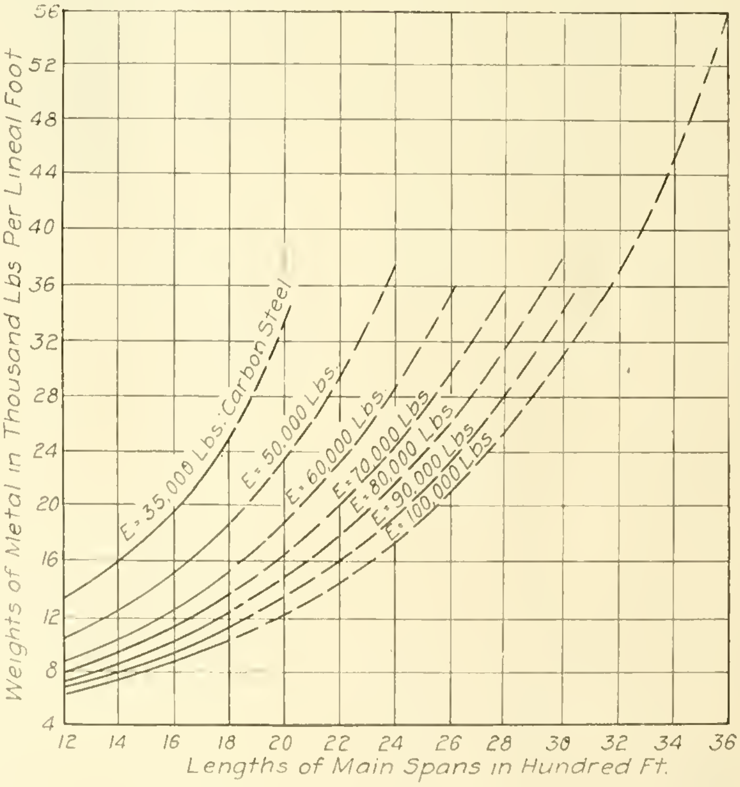


Fig. 7. Total Weights of Metal Per Linear Foot of Span for Double-Track, Cantilever Bridges of Carbon Steel and Alloy Steels of Various Elastic Limits When Impacts Are Assumed as Zero.

The increase of elastic limit to 50,000 or 60,000 lbs. per square inch will help greatly in the construction of spans of considerable length; but, for the very long spans now being planned, a still stronger material is needed, and it can economically be used at a very considerable increase in price. In the design for the 2,880-ft. suspension span for the North River Bridge at New York, there is great economy in placing the stiffening trusses along the cables, so that articulated joints are required, thus necessitating eye-bars in place of wire, which has heretofore been used in long suspension bridges.

The total live load on the eight tracks, two roadways, and two sidewalks is 20,000 lbs. per linear foot, when all are completely loaded; and, if the structure was made of carbon steel, the dead load would be about 110,000 lbs. per linear foot, which would make the sections almost prohibitive. The designs, therefore, have been based on the use of eye-bar cables of alloy, heat-treated steel, having an elastic limit of not less than 120,000 lbs. Such a steel has already been manufactured for limited sizes, and as it is only here required for eye-bars, from which any number of full-sized specimens can be tested to destruction, there will be no doubt as to whether the strength and other qualifications are fulfilled. With such material, the total average dead load of the structure is 64,000 lbs. per linear foot, so that this material makes a saving of 46,000 lbs. per linear foot, or practically 50 per cent of the total load on the structure.

Such a material will naturally cost a considerable price, but it will be about four times as strong as the usual carbon steel; and even at much more than four times the cost it would make a saving, on account of the large decrease in dead load.

About 40,000 tons of such bars will be required, practically all duplicates, so that there is little doubt that they will be furnished at a price which will make a great economy in the structure.

The writer therefore fully agrees with Mr. Waddell that such high-value alloy steels are a necessity for coming bridge structures, and he has the fullest confidence that our metallurgists will meet the demands as they arise.

DISCUSSION BY CHARLES EVAN FOWLER.

For many years the writer has known of the higher grades of steel produced by European manufacturers, but, owing to the great demand on the mills of the United States, it has been a case of taking what is offered by the manufacturers or paying the extras asked for nickel steel or anything except the ordinary grades. The tariff conditions will now allow the importation of European high-grade steels for Pacific Coast fabrication, but, until shops of sufficient size are established there, little advantage can be taken of such foreign products. This possible source of supply may eventually result in American mills being willing to meet the demand for nickel or other high steel alloys at a reasonable price.

Long spans are seldom contemplated or built from motives of economy, but are the result of the necessities of commerce, of finding good foundations, or as necessary connecting links in lines of traffic and communication, regardless of whether the structure in itself will be a paying investment. When long spans are found to be necessary the engineer must determine the class of structure that will be possible:

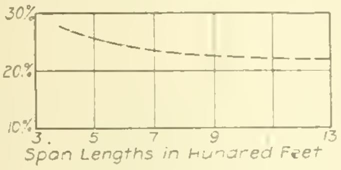


Fig. 8. Percentages of Carbon Steel in Simple-Span Bridges of Mixed Nickel and Carbon Steels.

(1) To carry the class and quantity of traffic to be imposed or cared for.

(2) That will be possible, due to the foundations that can be obtained.

(3) That will be possible, due to the material that can be obtained from which to fabricate it.

(4) That will be possible, owing to the limitations that may be imposed by the methods and means of construction.

(5) That will be a paying investment from a dividend-paying standpoint, if that is necessary.

(6) That will at least be possible from related financial conditions.

These factors are correlated to such an extent that a very wide investigation would be necessary, in order to set the limits of span and expenditure.

Recent researches seem to indicate:

(a) That simple spans can be constructed by using nickel steel to a greater or less degree, up to lengths of from two to three times those which have been built.

(b) That suspension spans and cantilevers will reach about the same cost at some span length between 1,600 and 1,800 ft.

(c) That, by the use of nickel steel to a large extent, suspension spans can be built economically, where the traffic is sufficient, up to about a 3,000-ft. span.

(d) That, by the use of nickel steel to a large extent, cantilevers can be built economically, where the traffic is sufficient, up to about a 2,500-ft. span.

However, inasmuch as the dead weight of such structures is very great in proportion to the live load, we can largely disregard impact, and use correspondingly high unit stresses, thus allowing at present cost the use of nickel steel for all the suspended structural parts of a suspension span, thereby making such a structure, with its high steel cables and comparatively low erection cost, the best to adopt for long spans, from every point of view, beyond a span length of about 1,700 ft. Such a structure will carry satisfactorily all classes of traffic, and satisfy most fully all the six requirements previously given.

The question of right of way and terminals may add so greatly to the cost of the project as to make its realization impossible.

The empirical erection cost formula, devised by the writer some years ago, contains six different factors which enter into the cost of erection of a bridge superstructure. This formula is:

$$C = a + \sqrt{l} + \frac{3}{4}h + \frac{200}{d} + 5p - \frac{1}{5}\sqrt{w} - 500$$

where $C =$ cost of erection, in cents per 100 lbs.;

$a =$ a constant for each type of structure;

$=$ 15 cts. for railway pin trusses;

$=$ 25 cts. for railway riveted trusses;

$=$ 20 cts. for railway girders;

$=$ 20 cts. for highway pin trusses;

$=$ 30 cts. for highway riveted trusses;

$=$ 15 cts. for highway girders;

$l =$ span length, in feet;

$h =$ height of falsework, in feet;

$d =$ daytime temperature, average, in degrees Fahrenheit.

Taking the case of a riveted railway span 225 ft. long, height of falsework 48 ft., average temperature 40°, 2 coats of paint, and weighing 2,100 lbs. per linear foot, we find the probable erection cost to be 83 cts. per 100 lbs., or $16.60 per ton.

A railway pin span of 144 ft., height of falsework 36 ft., average temperature 50°, 2 coats of paint, and weighing 1,400 lbs. per linear foot, would have an erection cost of 62 cts. per 100 lbs., or $12.40 per ton.

These two examples show what an influence a change in any one of the factors will have on the unit erection cost. The formula was deduced to fit certain conditions, which to a large extent were due to the personal equations of the designer and the erector; and, with plans prepared by some designers, the cost of erection would exceed very greatly the values found from the formula, it being only too common on the part of many designers to forget that structures must be erected at a reasonable cost, and still others seem to forget the process of erection entirely. Many erection costs, of course, will exceed greatly what they should, due to unforeseen causes.

DISCUSSION BY M. J. BUTLER.

It is an error to assume that the limit has been reached with ordinary carbon steel—undoubtedly the steel maker will supply a much better quality of steel if the customer is willing to pay the price, by

(a) fluid compression of the ingots;

(b) careful heat treatment and lighter and slower blooming, putting more work on the bloom;

(c) cropping away all segregated, pipes, and inferior metal; and

(d) careful re-heating of billets, slower rolling, and better work generally on the rolls and sections.

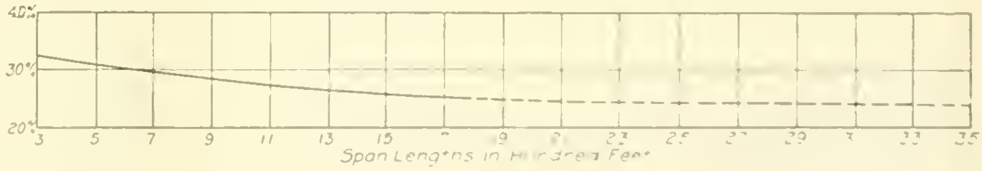


Fig. 9. Percentages of Carbon Steel in Cantilever Bridges of Mixed Nickel and Carbon Steels.

DISCUSSION BY L. J. LE CONTE.

The writer would suggest that the experimenters try a small batch of aluminum steel. The main objection to its use has been its high market price. Lately, however, the price has come down to 20 cts. per pound, in large lots. It is the writer's firm belief that if ordinary steel is first "thoroughly purified" and its elastic limit thereby raised to [illegible] lbs. per square inch, and then, say, 3 per cent of aluminum be added to the molten metal, the result will be a highly satisfactory alloy, and at no great increase in cost per pound. The aluminum not only strengthens the steel enormously, but also toughens it and [illegible] out all brittleness, and thus renders all general shop work much easier and more satisfactory and reliable in every way.

In case some of the material should be condemned as not coming up to the requirements of the specifications, the condemned material could easily be turned over to the wire works, which would be only too glad to get it, as this aluminum steel makes the finest kind of high-grade wire, suitable in every respect for suspension bridges, as it is exceedingly strong, tough, free from brittleness, and noncorrosive to a high degree. Ordinary sections of this alloy will have an elastic limit of 100,000 lbs. per square inch, and the wire possibly 200,000 lbs. per square inch; and at a cost probably not exceeding that of ordinary carbon steel by more than 2.5 cts. per pound.

DISCUSSION BY N. PETINOT.

The writer agrees with Mr. Waddell that the first step is to experiment on the purification of steel so that the metal may be brought to its maximum efficiency. He cannot see why it should not be possible to secure, regularly, physical properties within a limit of 5 per cent in a steel carrying predetermined quantities of carbon and nickel, for instance.

The speaker has been experimenting extensively, both in France and in the United States, with the manufacture of alloy steels, and particularly with nickel steels of the same supposed analysis, and has found that very often two melts of open-hearth nickel steel of the same analysis, when physically tested, have shown a variation of 15 per cent or more. Carrying the investigation further he found, by microscopic examination, a large difference between the two steels. The poorer very often showed very large colonies of manganese sulphides, iron silicates, and slag. These are weakening elements in steel of any grade, even if it is an alloy of plain carbon metal

When the manufacturers of steel find the way to remove such elements, it will be possible to secure the maximum physical qualities from each component of every grade of steel, and there should be no great difference between any two steels of the same chemical composition.

Increasing the carbon content in steel of any grade is a cheap way to increase the ultimate strength, and, according to the writer's experience, a satisfactory way, provided the steel has been thoroughly cleaned and the segregation reduced to the minimum.

In reference to the possibility of producing a nickel steel by the use of a ferro-nickel containing 10 per cent of nickel, this problem presents no difficulties at all, as such steels have already been produced in 5-ton melts. A few years ago a pig iron was produced by smelting pyrrhotite containing nickel in an electric furnace. This pig iron contained from 1 to 5 per cent of nickel and was low in sulphur and phosphorus. By varying the proportions of such pig iron in the mixture used in making nickel steel by the open-hearth process, it will be possible to obtain any nickel content actually desired.

It will be noticed that, in the making of nickel steel, the nickel is usually added with the cold charge, and does not, therefore, oxidize during the periods of melting and finishing the steel, and its use as a component of pig iron would be the same under similar conditions.

The writer believes that, by experimenting thoroughly with ferro-nickel in the manufacture of nickel steel, it will be possible to get a product which will be identical with that obtained by the use of metallic nickel, and at a much lower cost.

Mr. Waddell has asked what effect the use of titanium might be expected to have on the steel. Titanium has a great affinity for nitrogen and oxygen, so much so that, prior to this time, it has been very difficult to obtain a steel with a content of more than 0.25 of that metal, and such steel showed no particular advantages over one which had been treated with 0.10 of titanium, and after such treatment carried only 0.02 to 0.03 of the latter.

Titanium is on the market commercially in the form of ferro carbon-titanium, containing from 15 to 20 per cent of titanium and from 6 to 8 per cent of carbon. It has been found that a ferro-titanium, with a higher titanium content, has too high a melting point for general use.

Titanium, because of its affinity for nitrogen and oxygen, the latter either in the form of a gas or an oxide, is to be considered as a powerful deoxidizer and scavenger. It is the only deoxidizer known at present which can be used without danger of leaving any of the products of its oxidation in the bath of steel.

In the treatment of steel with titanium, the ferro carbon-titanium is always used as the last addition to the steel after it has been tapped into the ladle, that is, after manganese, silicon, or other alloys, if any, has been added. Titanium will react first on nitrogen (always present in a greater or less quantity), forming titanium nitrides, which rise to the top of the ladle. It will then act on the oxygen in solution in the steel, and on oxides of iron, manganese, etc.

Particular attention is called to the presence of manganese oxide in steel. Ferro-manganese is added to the steel, not only as a deoxidizer, but also to furnish sufficient manganese to combine with the sulphur, giving manganese sulphides, which are less brittle than iron sulphides. It has always been found that the rolling properties of steel are enhanced by a small manganese content, and it is a well-known fact that, by eliminating the manganese oxide completely, no blisters will be found when such steel is rolled into sheets and galvanized.

Another function of titanium is to reduce segregation to a minimum. The writer's experience has been that a clean product with very little segregation can be obtained by the use of titanium at an additional cost of from 25 to 50 cts. per ton of steel treated.

DISCUSSION BY LEON S. MOISSEIFF.

There is no doubt that the author points the right way into the future: there is a need of a material of a higher strength than that of the steel generally used in bridge building today. Whether it be nickel steel, as recently used in some of our long-span bridges, or another kind of alloy steel, which will still better meet the requirements of bridge building, there is a technical need of a higher strength steel. Not only would heavy railroad bridges with their increasing freight loads be benefited by the higher steel, but also highway bridges of the longer spans.

The advent of heavy automobile trucks has made it imperative to provide heavy floors for bridge roadways. To insure the good behavior of a pavement, not only a strong but also a stiff support is necessary. Such support will be furnished by a heavy timber floor with floor stringers at close intervals, or by reinforced concrete slabs. In either case a floor system will be required which will have considerable weight. To take care of this additional weight and load a high-strength steel is wanted.

Granting the desirability of the higher strength steel, questions of design may well be considered. One of these questions may be what will be the effect of the higher steel on the stiffness and stresses of bridges built of that material?

The higher elastic limit of the new steel will be utilized, of course, for allowing higher unit stresses. The coefficient of elasticity being practically constant for kindred steel, the higher unit stresses will result in greater elongations of the individual members and greater deflections of the entire structure. The increased deformation of the trusses means a greater deviation from the original truss-form, and will result in much increased secondary stresses.

Assuming (as the author apparently has in the paper) that the allowable unit stresses will retain the same proportion to their corresponding elastic limits as they have in the present practice in the case of carbon steel, the allowable stresses for the hoped for high-alloy steel of 100,000 lbs. elastic limit will be nearly three times those of carbon steel with its elastic limit of 35,000 lbs. Evidently the moving load deflections will become considerable and the secondary stresses may attain the rank of primary stresses. How, then, to take care of the secondary stresses in the erection of the bridge, and provide for the deformation of the trusses under moving load, will raise new and serious problems in bridge design.

Another question is how to connect efficiently the high steel members, apart from using pin connections. With common soft steel rivets the splices would be of excessive length, the efficiency of which is subject to much doubt. Consequently, high-steel rivets must be used. Nickel steel members have been designed by the speaker for corrsponding nickel steel rivets, but these rivets have not always proved very satisfactory. They are not easily well driven and are difficult to remove. This refers especially to field rivets which have to be driven with a gun or a machine of limited pressure. Now, if nickel steel rivets having an elastic limit of 50,000 lbs. present some difficulties, what will rivets of, say 80,000 lbs. elastic limit offer?

DISCUSSION BY F. W. SKINNER.

The writer, having recently had an opportunity of seeing some phenomenally large carbon steel rivets, made for the new Quebec Bridge, would like to confirm to some extent the views of the author and of Mr. Hodge. Some of these rivets—those for the cantilever arms—were 7 ins. or more in length, with a diameter of 1¼ in.

Some experimental work was done in which these rivets were specially designed with a slight taper at the points. They were quenched at the points in cold water. When driven, they were held in place by a pneumatic bucker, and the driving was done with a pneumatic hammer. The pneumatic bucker was a special tool, having a hollow cylinder in which a hammer was arranged. The hammer and cylinder were controlled independently, which enabled the bucker to be held in position, and then when the ordinary hammer was operated on the point of the rivet, the secondary hammer in the bucker was operated, and the rivet was thus driven simultaneously from both ends. The result was that the rivets were exceedingly well driven. A row of rivets was driven partly by yoke machines and partly by the hammer and bucker process, as in the field. When they were cut in two, it was almost impossible to distinguish between them, the field-driven rivets being, for every practical purpose, as good as those driven by machine in the shop. Therefore, it seems quite reasonable to expect that field-driven rivets can be made substantially as good as shop rivets, and, if nickel steel rivets can be driven satisfactorily, it may go a long way toward producing equally good results in the field.

General

Hydroelectric Power Station of the Northern Ohio Traction & Light Co.

Contributed by Jay C. Lathrop, Mem. Am. Soc. C. E., Baltimore, Md.

The Northern Ohio Traction & Light Co. have recently completed a hydroelectric development on the Cuyahoga river near Akron, O., which has many interesting features.

It is of historical interest that 80 years ago, in 1833, a certain Dr. Crosby promoted a hydraulic raceway, something over four miles long, to utilize the flow in this portion of the Cuyahoga. He succeeded in interesting eastern capital to such an extent that over $500,000 was spent in this work. The raceway was built along the side of the river gorge, and for a great portion of the distance the water was retained by means of an enormous masonry wall, built of stones of such size, that it is difficult to understand how they were handled with the methods in use at that time. This raceway was nearly completed, and water had been turned in and then off, so that leaks which had developed might be repaired. Then the panic of 1837 occurred and stopped all work. Before the company recovered from the effects of the panic the use of steam for power had become so common that water power was considered obsolete. If the $500,000 had been invested at that time at 4 per cent, it would now amount to nearly $10,000,000—more than enough to buy all of the stock of the Northern Ohio Traction & Light Co. It may be noted that a small portion of this work was utilized to advantage during the construction of the present plant.

The principal reason for the present development at this time was the fact that the company was building a 40,000 kilowatt steam power station, and owing to the scarcity of water in this river at the low water stages, found it necessary to construct a dam to form a condensing water pond about 40 acres in extent. The general layout of the steam plant, pond, highway, electric-railway bridge, dam, pipe-line, hydraulic power station, and transmission lines is shown on the accompanying plan, Fig. 1

At the point where the highway crossed the river it was necessary to raise the highway bridge about 30 ft. In doing this it was found desirable to replace the old steel structure. New concrete abutments were built upon the old stone abutments, and a second-hand Pratt truss railway bridge was bought and erected. Extending the supports in this manner proved to be a costly mistake, for when the fills forming the approach to the bridge were made, the weight of this material crushed the old stone and allowed the bridge to settle. This made it necessary to build false work under the truss, and reinforce the abutments. In addition the filling was removed from behind the south support, and replaced by a 75-ft. through-plate girder span.

Raising the level of the water in this manner also made it necessary to protect the steel trestle towers which carried the electric railway tracks. This was done by encasing them in solid concrete up to a point 3 or 4 ft. above the usual water level. A portion of this trestle was of wooden construction. This was replaced by a steel trestle of modern design.

Before beginning work on the dam, pipe line and hydraulic power station, a construction track about a mile in length was built along the old race mentioned above. All material of every description was delivered over this track (from the company's siding on the Pennsylvania R. R. just above the site of the steam power station) by the company's electric work car.

The dam shown in the illustrations Figs. 1 and 2 is about 450 ft. wide at the top and has a height of about 52 ft. from the bed of the stream to the top of the spillway. Near the bed of the stream under the spillway, four 48-in. pipes were placed provided with gate valves which may be opened should it ever be desirable or necessary to empty the reservoir. Since its construction this dam has passed through the unprecedented flood of March, 1913, without any apparent damage.

Water is delivered from this pond to the turbines through a 90-in. steel conduit 2,760 ft. long. This conduit was built on concrete piers, Fig. 4, and is laid on grades varying from 3.25 ft. to the hundred just below the dam, to zero for the last 100 ft. This conduit is built along the side of the Cuyahoga River gorge as shown by the accompanying plan. Near the upper end of this pipe, just below the dam, a 90-in. motor operated gate valve was installed. Midway between the dam and power house a surge tank 60 ft. high and 10 ft. in diameter was provided. In the length of 2,760 ft. five expansion joints were made. The pipe line terminates in a receiver 8 ft. in diameter which has three 51-in. connections for the turbines. At the lower end of this receiver another surge tank was built 4 ft. in diameter and 125 ft. high. Two manholes are provided in the receiver.

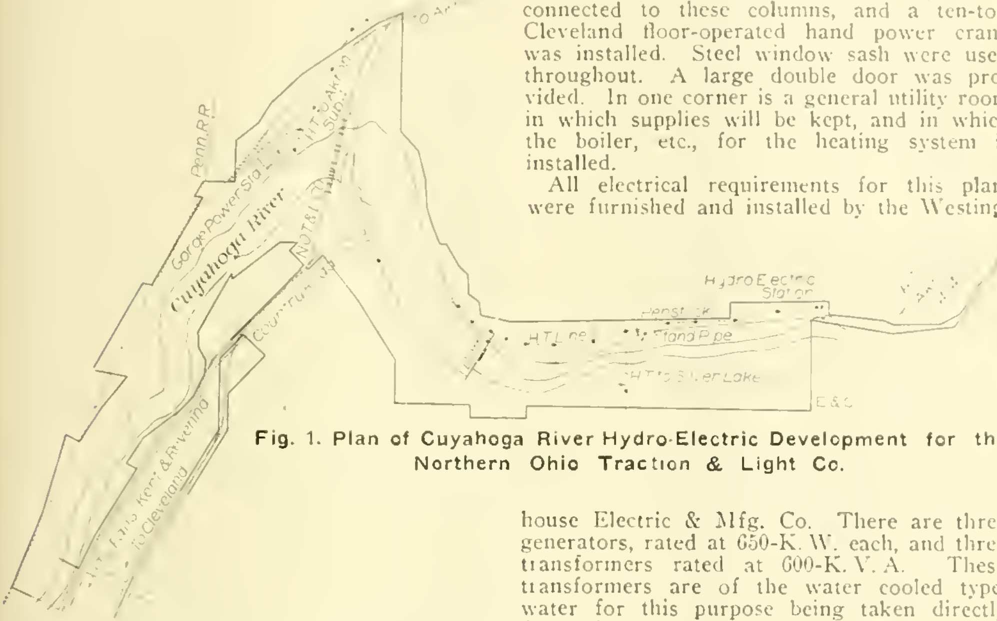


Fig. 1. Plan of Cuyahoga River Hydro-Electric Development for the Northern Ohio Traction & Light Co.

The thickness of metal used on the pipe line is as follows: Starting from the upper end—750 ft 5/16 in. thick; 690 ft. 3/8 in. thick; 630 ft. 7/16 in. thick, and 690 ft. 1/2 in. thick.

Fig. 2. Dam Under Construction, Cuyahoga River Hydro-Electric Development.

Fig. 3. Dam Nearly Completed, Cuyahoga River Hyrdo-Electric Development.

The receiver is also made of 1/2-in. metal. Soft basic open-hearth steel was used for all the above work. All circular seams are single riveted, while all longitudinal seams are double riveted, the sizes of the rivets varying from 3/4-in. at the upper end to 1-in at the lower end.

The hydraulic equipment consists of three 750-HP. Trump horizontal turbines which are direct connected to the generators by means of a flexible coupling. These turbines are carried upon two 15-in. 60-lb I-beams, which also support the generators. These beams are buried and are anchored to the foundation by means of heavy anchor bolts.

Governors of the oil pressure type were furnished by the Woodward Governor Co. These governors are motor operated.

The power house foundations are placed upon solid rock. These foundations, as well as the floor, are of concrete. The exterior walls are of No. 2 paving brick laid in English bond. The roof consists of a concrete slab upon which is laid red Spanish tile. This roof is supported by six Fink roof trusses which in turn are supported by steel columns consisting of a 12x3/8-in. webb plate, and four 3x3x3/8-in. angles. Crane runways were connected to these columns, and a ten-ton Cleveland floor-operated hand power crane was installed. Steel window sash were used throughout. A large double door was provided. In one corner is a general utility room in which supplies will be kept, and in which the boiler, etc., for the heating system is installed.

All electrical requirements for this plant were furnished and installed by the Westinghouse Electric & Mfg. Co. There are three generators, rated at 650-K. W. each, and three transformers rated at 600-K. V. A. These transformers are of the water cooled type, water for this purpose being taken directly from the conduit. One 40-K. W. exciter is belted to one of the main generator shafts, and one 40-K. W. motor generator set was installed. One 10-K. V. A. transformer was installed for lighting the building, and one six-panel switchboard, with suitable lighting arresters, were also furnished by this company.

This station is tied electrically into the 22,000-volt high-tension line, which runs along the Cuyahoga gorge to Akron. It will be operated in parallel with the main steam station shown on the accompanying plan.

Numerous difficulties were encountered in all parts of the work, mainly due to the precipitous nature of the sides of the gorge. Although the work was started in 1911, the final details in the power station and pipe line have only recently been completed. The dam, the concrete benches supporting the pipe line, the pipe line, the power house, and the various mechanical installations were all executed by different contractors and by a variety of different methods.

As stated heretofore, the construction track along the old raceway, was used in handling all materials for all parts of the work, and this led to considerable confusion, as at times there were half a dozen different jobs going on at one time.

The dam was started in 1911, and materials were handled by means of a cable way, stretching entirely across the gorge. The concrete for this dam was distributed from a tower through chutes. This is very well shown in one of the accompanying photographs. In spite of a severe flood in October, which did considerable damage to the upper portion of the deck, the dam was completed in the latter part of November, 1912.

The contractor who built the concrete benches for the pipe line used a concrete mixer, mounted on a car, and ran the concrete from this car, which traveled on the construction track, through chutes to the work.

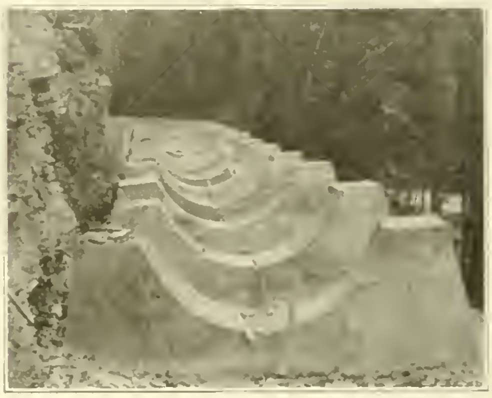


Fig. 4. Concrete Piers for 90-in. Steel Penstock, Cuyahoga River Hydro-Electric Development.

The contractor for the pipe line simply rolled the sections of the steel pipe off the car and let them roll down the gorge. Some of these sheets were buried so deep by slides during the floods of March, 1913, that it required considerable excavation to reclaim them. The steel pipe was then erected by the standard method of the contractor.

The power house site was something over 100 ft. below the level of the construction track and various methods were adopted by the contractors to lower materials to the building site. The concrete was mixed at the construction track elevation, and run through a chute to the work. The bricks were rolled down the bank, while machinery of various kinds was placed on heavy wooden sleds, and lowered by a steel cable run through sheaves, and attached to a work car on the construction track.

What will probably amount to the largest yardage of concrete highways ever built in a single year within a radius of 40 miles was started in May when Milwaukee, Sheboygan, Racine, and Kenosha Counties, Wisconsin, commenced work on 52 miles of concrete roads.

A scheme is said to be under consideration for the establishment of an automatic transport service across the Andes to compete with the Trans-Andean Railway.

Dam Foundations.

To the Editors:—Kindly permit me to make some comments on the letter by Prof. H. P. Boardman in your issue of May 27 on the subject of dams.

Prof. Boardman quotes my statement that "no dam ever failed that was heavy enough and wide enough at the base to be stable under the pressure of water on the wetted side and underneath it, regardless of what it was founded on." He seems to interpret my meaning to be that dams ought to be built regardless of what they are founded on. But there is no need to take this meaning. My letter was written to contrast the importance of *design* as compared with *foundation*—to emphasize the fact that failures that have been commonly and uniformly attributed to foundation are in reality due to faulty design, and there is no reason to anticipate stability in a dam when the very design does not take into account the possible pressure on it.

A reinforced concrete structure might be so fragile that it would fail by reason of a very slight settlement in one of the footings, and investigators might blame the whole failure on the foundation. But what about a design that lacks reinforcement to give the structure toughness? Why blame a foundation for doing the thing that all foundations, not of solid rock, always have done and always will do, namely, settle a little? Why blame the soil under a dam for doing what soil always has done and always will do, namely, wash away when a stream of water under heavy pressure flows over it? When the pressure under a dam exceeds the weight of the dam, it will lift the dam and allow a stream of water under heavy pressure to flow over the soil, if the dam itself is not carried away. This erodes the soil and undermines the dam. If the dam had been heavy enough to keep its bed, the water would not have created a cavity and started a flow. In hollow dams like the Stony River and the Pittsfield, because of the reinforcement, the superstructure is apt to hold together, for a time at least, until the base is undermined. In mass concrete dams of insufficient width of base, like the two Austin dams, the wall breaks up and is at once pushed down-stream by the impounded water.

"Facts" have been called for on this subject. The quotation from my letter, in the second paragraph of the present letter, is a fact, and a fact of some significance. There are dams as wide as 0.85 of the height, but none of them have ever failed. The solid masonry dams that have failed have had bases about 0.65 of the height, for this is the theoretical width for dams designed regardless of under pressure. Is it throwing away money to make the width of base 30 per cent more and make something that water cannot lift? This does not mean 30 per cent added to the cost of a dam, for a dam of proper width does not of necessity require the cut-off walls that have "safeguarded" the life of so many dams (most of the failures have one or two of them). Furthermore, the cost of a dam is not directly proportional to the amount of masonry. In any event, would it not be good engineering to spend even 30 per cent more for a dam if this means the difference between failure and safety?

Here are some more facts. They are taken from "Concrete Age" for May. A concrete tank at the foot of Walrous Ave., Tampa, Fla., 10 ft. square and 13 ft. deep, was made in the ground, and the earth was tamped around it. The walls were 13 ins. thick and the bottom was 16 ins. thick. Its weight was then about 90,000 lbs. This heavy tank was completed on Saturday and was lifted over Sunday by water that seeped beneath it, and *raised 47 ins.* It would take just about 13 ft. of head of water to lift this tank if it were entirely free of friction on the sides, and if the water had unhindered access to the entire area of the base. Are these not the very facts that your editorial in the issue of April 1, p. 376, states that the profession needs? On page 600 of the issue of May 27 the same need is reiterated.

Other facts along the same line are these: When water rises over ground on which there are boards lying, even though these be embedded well in the ground, these boards will float away. Now watersoaked wood weighs more than two-thirds as much as the same volume of water, hence the pressure on the under side of these boards must be more than two-thirds of the full head on the under side of the boards before they can begin to float. Besides this the water pressure must overcome adhesion to the mud as well as friction on the sides of the boards. This would indicate clearly that water acts as though it had free access to the under surface of the boards. Why, then, will it not act in the same manner on the under side of a masonry dam in contact with the soil? This would be still more true of a dam founded on rock for the joint between rock and rock is more readily accessible to water than that between clay and rock.

Prof. Boardman states that "in a general way" it is a crime to design a dam without considering upward pressure, and then he takes all of the force out of his assertion by saying that all of the "considering" that is necessary is to take a good look at the foundation and squirt in some grout, then make a cut-off wall that will cut off. But underground water is not controlled by such simple means as this would seem to call for. Very often concrete tanks that are built with care to retain water, built up in the air with every advantage of easy access, hold that water like a sieve. When engineers want to make sure of water being kept out of the basement of a building they usually paper the entire ground with several thicknesses of tarred felt. Did you ever hear of a dam builder papering the entire bottom of a lake or stream? Cracks, both vertical and horizontal, in a concrete wall are very common. What if a crack should develop just above a carefully examined and grouted foundation? What if the cut-off wall should crack away from the main dam?

Prof. Boardman mentions the Hauser Lake dam as another failure due to foundation. I have four letters in my files in which I warned the engineer of this dam during its construction of the danger of the *design*. It happened that I was consulted concerning features of the steel deck. I looked farther than the steel deck and found that a high apron exerted an unresisted uprooting pull on a row of sheet piles at the foot of the slope of the steel deck. In order not to be connected with the failure, which I anticipated, I did what I could to avert it. This dam failed exactly as I predicted it would, but my warning was ignored.

Here are some more facts that have a bearing on under pressure in dams. In "Engineering News," June 4, 1914, there is a description of an earth dam that failed on the upper Sevier River, in Utah. It is stated that "large sections of the lower face began to slip off." Evidently the water was just beneath this lower face, the hardened compact surface of the down-stream slope. The theory is here set forth that most failures of earth dams are one piece with those of masonry dams. The under pressure in the earth dams is exerted by a sort of semi-liquid mud that is formed by the saturation of the interior of the dam. The natural and apparently reasonable thing to do is to compact the down-stream slope, sometimes to pave and grout it. But this is the very thing that is apt to cause the entire dam to fail. Instead of the water near this surface being allowed to escape, thus drying out the interior of the dam, it is held in until the saturation of the interior is complete. This semi-liquid mud then exerts a hydraulic pressure on the hardened crust and flows out, resulting in failure of the dam. A full height core wall, and little or no compacting on the surface would seem to be the remedy. Possibly the planting of grass would be a great aid, as this helps to retain the porosity of soil. Also roots reach down and take up moisture from below the surface. I think this would make a good subject for discussion by engineers engaged in building earth dams.

Here are some facts that substantiate my theory. In "Engineering Record," April 29, 1911, there is a description of the failure of the earth dam of the Julesburg reservoir. It was clearly evident that this dam failed by upward pressure of the water, and the article so states. The overlying mass of rock and earth on the toe of the slope was lifted by the pressure of the water. The remedy here applied was to introduce a core wall in the rebuilding.

In the same issue of the "Record" there is a description of another dam that failed. This was a hollow dam of mass concrete filled with sand. It was very broad in section, nearly like an earth dam. But when water was let in the dam failed. Evidently the failure was by access of water to the sand interior and pressure that lifted the down-stream layer of concrete.

Prof. Boardman says, "If there were a stratum of sand below the base of such a heavy dam and the water in the reservoir gained access to it and also gained exit below the dam, it would probably not take any longer to undermine the structure than if it were an earth fill or a hollow concrete dam. The result would be, no doubt, a worse wreck than occurred at the Pittsfield dam 'wash under.'" He also cites failures in levees which started by "seeps" or "high water springs." In making the above statement and these citations Prof. Boardman fails to consider the fact that in a properly built earth dam there is a great barrier against the flow of water in the core wall which does or should prevent saturation of the soil under the down-stream slope: this is not found in the levee. In the "heavy" masonry dam the barrier is the broad base of the dam and pressure that resists the uplift of the water: the hollow dam does not have this pressure, hence the water lifts it and forms its own cavity where the exit is free. It is against all reason that under the broad bases of the hollow dams that have failed the water could inaugurate a course of free flow that would carry away the soil, unless it could first lift the overlying slab. Frozen weep holes would be exactly the condition that would enable the water to lift this light shell, and both of the dams already referred to failed in midwinter.

Springs in soft soil have existed for generations, but they carry little or no sediment. It is when a saturated sub-soil under a hard crust breaks through that crust that trouble may be expected.

Yours very truly,
Edward Godfrey.

Monongahela Bank Bldg., Pittsburgh, Pa.,
May 9, 1914.

Cement Products Exhibition.—Announcement has been sent out by the Cement Products Exhibition Co., 208 South La Salle street, Chicago, that the Eighth Annual Chicago Cement Show will be held in the Coliseum, Feb. 10-17, 1915.

Only one Cement Show will be held in 1915. The time has been shortened and arrangements have been made to give the exhibitors more time both in installing and removing exhibits.

A new floor plan has been developed which, it is believed, will meet with universal approval. A greater number of spaces will be provided and it will be possible to give the main avenues an attractive appearance. A uniform installation plan, without high partitions, will be adopted. The management will again prepare the exhibit spaces so that no attention on the part of the exhibitors will be necessary other than to place their exhibits in position.

It is expected that the floor plans, application blanks and complete information will be sent out from the offices of the Cement Products Exhibition Co. to the list of respective exhibitors not later than August 1.

City Manager Waite of Dayton, Ohio, has awarded a contract for approximately 1,170 meters to the Union Water Meter Co. of Worcester, Mass.

Water Works

The Evolution of the Water Tower of the Maryville, Mo., Water Works.

Contributed by Clifford F. Phillips, Consulting Hydraulic and Sanitary Engineer, St. Louis, Mo.

The first water works standpipe at Maryville, Mo., was built in 1886. This standpipe was a simple circular cylinder of sheet steel built in accordance with the general practice at that time. On Feb. 28, 1893, this standpipe failed. A number of theories were advanced as to the cause of its collapse. The accepted one is that the standpipe had been kept full during the extremely cold weather that preceded the failure, so that it froze over solid. About Feb. 27 the weather moderated. For some reason or other, probably due to bursted service pipes, the water was drawn away from the standpipe faster than it was pumped in, until it was only about half full. On Feb. 28 it was warm enough to melt the ice frozen over the top sufficiently to allow it to slide down. This immense body of ice struck the surface of the water with sufficient force to burst the pipe wide open.

In the same year, 1893, the water company set to work to replace this standpipe with a tower that they figured would be practically everlasting. A photograph of this tower is shown in Fig. 1. This tower was not built very long when it began to show vertical cracks extending from the first window from the ground to the second, which is on a level with the steel tank inclosed. This tower cost in the neighborhood of $25,000, all brick and stone work being laid in cement mortar.

Referring to Fig. 2, a sketch of section of tower shown in Fig 1, it will be noticed that the edge of the steel tank is 2 ft. from the veneer wall or 1 ft. 3 ins. from the 18-in. supporting wall. The support for the tank was a hemispherical brick arch. Had this arch been made a reinforced concrete slab, so that the resultant forces would have been vertical, these cracks would never have appeared. Immediately after the cracks were noticed steel bands were placed around the tower at the point of greatest bulge. The tank was kept half full of water so as to lighten the load as much as possible. Observations made from time to time showed that even with half a load the cracks were continually widening. At the time the writer inspected the tower in January, 1913, daylight could be seen through the four vertical cracks located in line with the windows.

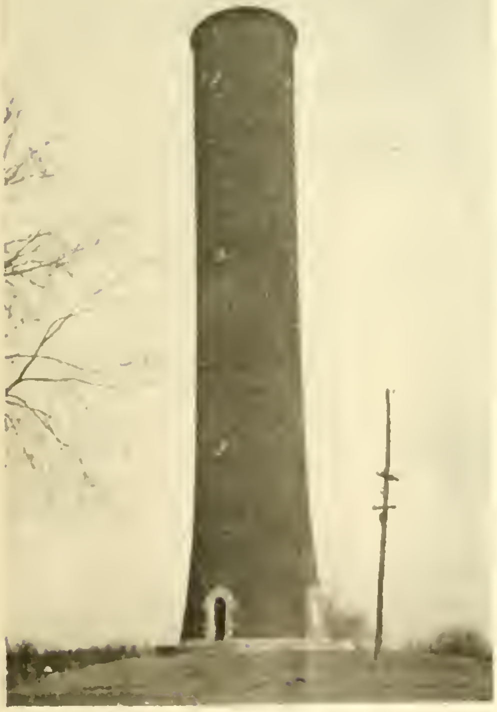


Fig. 1. Exterior View of the Brick and Steel Water Tower of the Maryville, Mo., Water Works as Built in 1893.

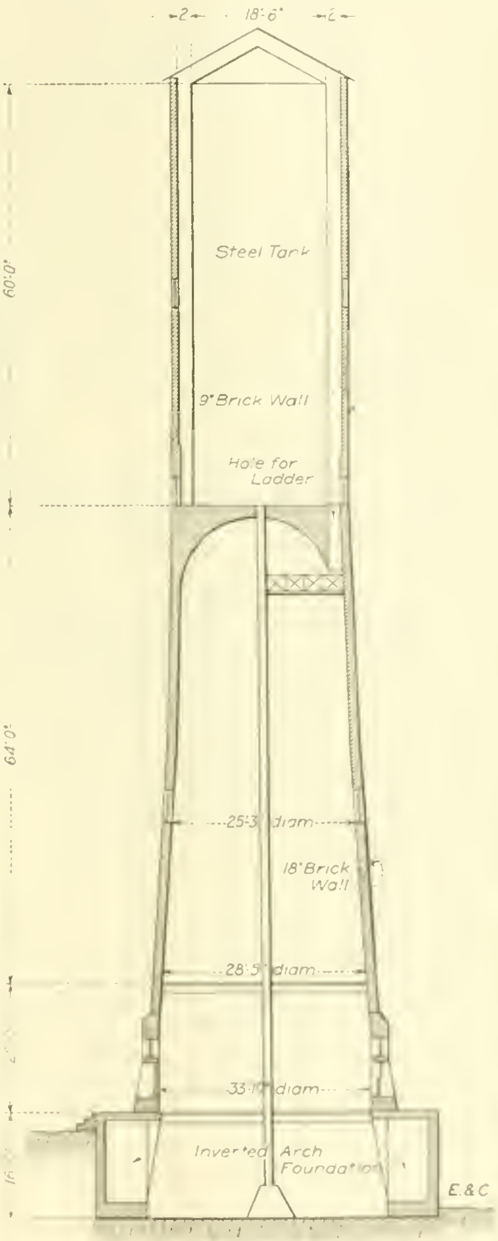


Fig. 2. Sectional Elevation of the Brick and Steel Water Tower Built at Maryville, Mo., in 1893, Showing Hemispherical Brick Arch Support of Steel Tank.

In 1912 the city of Maryville voted $100,000 to build a new water plant or buy the old one. Various appraisals were made of the old plant and the water company finally agreed to take $50,000 for their property. The problem then presented itself to the city, as to what could be done with the old tower. They wished to retain the old tower exterior, if possible, and put the enclosed steel tank on steel legs supported on a foundation independent of the brick tower. From an architectural standpoint, the exterior of the structure was very pleasing to the eye, so that it was agreed to design a new supporting tower inside of the old. The amount of work and the hazard in removing the brick arch and supporting the old steel tank was so great that the writer's estimate was over $9,000. Since this was more than the city wished to spend for this work they agreed to take bids on the design they desired and also alternate propositions to be proposed by the contractors. The bids on the design, in accordance with the city's wishes, were very close to the estimate, so it was decided to abandon this scheme and adopt some design that proved more economical and more efficient, if not so beautiful. After several weeks of considering various designs offered, the design of the Chicago Bridge & Iron Works was adopted. A photograph of the finished structure is shown in Fig. 3.

The work consisted essentially of removing the old steel tank and tearing down the brick work level with the first window above the ground, which point is 50 ft. above the finished grade around the tower. A concrete beam was then placed on the top of the wall to support the steel tower posts. The opening was covered over with a reinforced concrete slab. On this foundation a steel tower and tank 61 ft. from the lower side of beam to the eaves of the tower roof was placed. The depth of the hemispherical bottom of the tank is 13 ft. 4 5/16 ins. and the depth of the cylinder 27 ft. The diameter is 26 ft. 8 ins., making a tank of 150,000 gals. capacity. This tower was completed and put in service in Feb., 1914.

A complete new tower and tank would probably not have cost any more than this structure, but in this way the portion of the old tower that was dangerous was removed and the new one was placed on an excellent foundation.

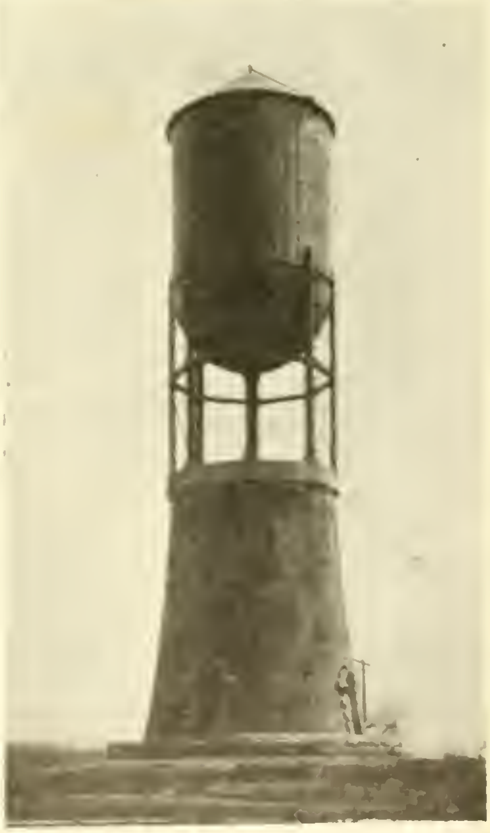


Fig. 3. View of Water Tower of Maryville, Mo., Water Works as Remodeled in 1914.

Convention of City Managers.—City Manager C. E. Ashburner of Springfield, Ohio, is sending out invitations to city managers throughout the country to meet in convention at Springfield, August 4, 5 and 6, next.

Data and Discussion on the Cost of Constructing and Operating Slow and Rapid Sand Water Filtration Plants.

At the first of the present year the total population in the United States supplied with water filtered by slow sand filters was 5,398,000. The corresponding figure for rapid sand filters was 11,893,000. About 69 per cent of the total population of this country supplied with filtered water is served by rapid sand filters. Both forms of filtration are, of course, well established but it appears to the spectator that the slow sand process is decidedly on the wane in this country. For certain waters and under certain local conditions affecting the cost of building and operating works, however, many of our leading filtration engineers would not hesitate to recommend the adoption of the slow sand process. While questions of cost should not in themselves govern the manner of handling problems affecting community health and comfort, such considerations will usually prevail in the choice of a filtration process. The present article summarizes the most notable recent discussion on the cost of constructing and operating the two types of filtration works.

In a very extensive paper before the recent annual convention of the American Water Works Association Mr. George A. Johnson of New York City discussed present day water filtration practice from all points of view. We here republish the portion of his paper bearing on the relative cost of slow and rapid sand filtration. Following the excerpt from Mr. Johnson's paper we publish a discussion of the same subject by Mr. John H. Gregory of New York City, which was delivered before the same convention. Mr. Johnson's statement as to cost follows:

RELATIVE COST OF SLOW SAND AND RAPID SAND FILTRATION.

Construction.—In discussing the cost of building water filtration works of the slow sand and rapid sand types, respectively, consideration will be given only to those items referring to the filter plant proper. Cost of land, pumping machinery, outside connecting piping, intakes, etc., in fact everything outside the filtration plant proper, will not be considered.

For slow sand filter costs the items will include the necessary filter buildings and filters with all appurtenances, all inside piping, sand handling apparatus, preliminary sedimentation basins, preliminary filters and appurtenances and clear water reservoirs.

For rapid sand filter costs the items will include the filter buildings and filters with all appurtenances, all inside piping, filter washing apparatus, coagulating and clear water basins. Thus a fairly good idea may be had of the relative cost of building purification plants of the two types.

It is true that, on account of the much greater area required, the cost for land is far greater in the case of slow sand filtration systems than for rapid sand systems. Roughly, other things being equal, land will cost 20 times as much for a slow sand filter installation as for a rapid sand plant. Furthermore, in large projects, it is often difficult conveniently to locate a site for slow sand filters, while for a rapid sand filter plant it is a relatively easy matter as a rule. If it is necessary to go a long distance in locating an extensive and suitable area of land for a slow sand filter site there is incurred a large expense for a conduit to bring the filtered water to the city. This is very rarely necessary in the case of rapid sand filter projects. So that, in studying the comparative figures which follow, it must distinctly be borne in mind that the costs given for slow sand filter installations are really low, since the important considerations just mentioned are not charged against them.

The figures given in Table I show that the approximate relative cost of building the slow sand and rapid sand filter plants mentioned was $32,600 and $12,100, respectively, per 1,000,000 gals. daily capacity. At 5 per cent the fixed charges on these sums would amount to $4.47 and $1.66, respectively, per 1,000,000 gals. of water filtered.

Operation and Maintenance.—The cost of operation and maintenance of filtration plants in a large measure varies, of course, with the quality of the raw water. In a general way the following examples will serve to show the charges ordinarily made against the operation and maintenance of representative water filter plants in this country.

To summarize, the average cost of building seven of the largest and most modern slow sand filter plants was $32,600 per 1,000,000 gals. daily capacity; and, likewise, the average cost of building six of the largest, and two medium size, rapid sand filtration plants was $12,100 per 1,000,000 gals. daily capacity. The average cost of operation and maintenance varied widely, of course, but averaged $2.86 and $4.04 per 1,000,000 gals. of water filtered by the slow sand and rapid sand systems, respectively. Adding these last figures to the fixed charge on the first cost of construction makes up the following totals:

	Per 1,000,000 gals.
Slow sand filtration............	$7.33
Rapid sand filtration............	5.70

TABLE I.—COST OF CONSTRUCTION OF SLOW SAND AND RAPID SAND WATER FILTRATION PLANTS.

City.	Kind of filters.	Present daily filtering capacity, gals.	Approximate cost per 1,000,000 gals. daily capacity.
Albany, N. Y..............................	Slow sand	20,000,000	$20,000*
Pittsburgh, Pa.	Slow sand	200,000,000	26,000*
Philadelphia, Pa.:			
Torresdale	Slow sand	250,000,000	37,700*
Upper Roxborough	Slow sand	28,000,000	29,800
Lower Roxborough	Slow sand	17,000,000	26,300*
Belmont	Slow sand	60,000,000	45,200*
Washington, D. C..........................	Slow sand	100,000,000	30,000†
Cincinnati, Ohio	Rapid sand	112,000,000	11,400‡
Columbus, Ohio	Rapid sand	30,000,000	13,000§
Dallas, Texas	Rapid sand	15,000,000	13,000
Harrisburg, Pa.	Rapid sand	16,000,000	10,300
Little Falls, N. J.........................	Rapid sand	32,000,000	15,000
Lorain, Ohio	Rapid sand	6,000,000	14,000
New Milford, N. J.........................	Rapid sand	24,000,000	11,000
Watertown, N. Y...........................	Rapid sand	8,000,000	11,250
Weighted averages	Slow sand	....	$32,600
	Rapid sand	....	12,100

*Cost of preliminary filters included. †Cost of Dalecarlia Reservoir not included. Cost of McMillan Park Reservoir included, and also cost of remodeling Georgetown Reservoir, as well as cost of coagulating basin. ‡Cost of large plain sedimentation basin not included. §Cost of softening works not included.

DISCUSSION BY JOHN H. GREGORY.

Mr. Gregory's discussion of the paper prepared by Mr. Johnson is here given in full:

The author has presented a paper which contains much of interest to water works engineers and superintendents interested in filtration, and there is much in the paper with which the speaker is in hearty accord. There are, however, some phases of the subject on which the speaker holds somewhat different views from those of the author, among which may be mentioned the questions of the cost and difficulty of securing sites for slow sand filters and those of the cost of both slow and rapid sand filters.

TABLE II.—COST OF OPERATION AND MAINTENANCE OF SLOW SAND AND RAPID SAND FILTRATION PLANTS.

Year.	City.	Kind of filters.	Average volume of water filtered daily. Gals.	Cost of operation and maintenance per 1,000,000 gals. of water filtered.
1911	Albany, N. Y.........................	Slow sand	20,000,000	$2.50
1912	Pittsburgh, Pa.	Slow sand	100,000,000	3.41
1911	Philadelphia, Pa.	Slow sand*	9,000,000	5.62
1911	Philadelphia, Pa.	Slow sand†	13,000,000	3.59
1911	Philadelphia, Pa.	Slow sand‡	38,000,000	3.88
1911	Philadelphia, Pa.	Slow sand§	202,000,000	1.91
1912	Washington, D. C.....................	Slow sand	62,000,000	4.01
1912	Cincinnati, Ohio	Rapid sand	50,000,000	4.12
1911	Harrisburg, Pa.	Rapid sand	9,000,000	3.93
1912	Little Falls, N. J....................	Rapid sand	30,000,000	3.20
1912	Louisville, Ky.	Rapid sand	25,000,000	3.48
1912	New Orleans, La......................	Rapid sand	16,000,000	6.32
Weighted average		Slow sand	....	$2.86
		Rapid sand	....	4.04

*Lower Roxborough; †Upper Roxborough; ‡Belmont; §Torresdale.

Selection and Cost of Site.—The author states: "It is true, on account of the much greater area required, the cost for land is far greater in the case of slow sand filtration systems than for rapid sand systems. Roughly, other things being equal, land will cost 20 times as much for a slow sand filter installation as for a rapid sand plant." There is but little question that, under ordinary conditions, the cost of land for slow sand filters will exceed that for rapid sand filters, owing to the larger area required, but that it will amount to as much as "20 times" that for slow sand filters is open to question. Those who have had experience in acquiring land for water works projects know that, even if only a small piece of land is required on which to locate the works but that this piece of land is a portion of a much larger tract, the purchaser might often as well buy the whole tract of land as to buy only a small portion of it. Thus it may readily be that, in acquiring a site for rapid sand filters, much more land would be actually purchased than needed simply for the plant. It may be that the area required for the construction of a slow sand filter plant will be 20 times that required for a rapid sand filter plant but that the land will cost 20 times as much does not necessarily follow.

In discussing further the question of site the author states: "Furthermore, in large projects, it is often difficult conveniently to locate a site for slow sand filters, while for a rapid sand filter plant it is a relatively easy matter as a rule. If it is necessary to go a long distance in locating an extensive and suitable area of land for a slow sand filter site there is incurred a large expense for a conduit to bring the filtered water to the city." The inference might be drawn from this that it has been exceedingly difficult to secure sites for large slow sand filter plants but the history of many of the plants which have been built will hardly bear out this inference.

In commenting on some of the plants which the author mentions in Table I the speaker might say that he has been personally connected with 7 out of the 15 plants mentioned and is familiar with the reasons which led to the selections of the sites on which these works were built. It is, of course, necessary to have land on which to build filters, whether of the slow or rapid sand type, but in the selection of a site local conditions are often a very important factor.

At Albany, N. Y., for example, practically the first ground available on which filters of either type could be built and which could be purchased at a reasonable price, was that on which the slow sand filters were actually built, just north of the city line. It happened that there was another tract of land a little nearer to the existing water works pumping station, to which the filtered water was to be delivered, where it would have been possible to build either slow or rapid sand filters. To have acquired land there, however, for either type of filters, would have been very expensive. The site on which the filters were built had another advantage in that the intake there would be further away from local sources of pollution. At Albany the Hudson River is tidal. The sewage from the city was and still is discharged into the river without treatment and on flood tides there is often an upstream current which carries some sewage with it. It will be seen then that other factors than simply area of land alone may have a bearing.

At Pittsburgh, Pa., as far as the speaker is aware, no difficulty was found in securing a site for the slow sand filters there, which plant is one of the largest in the world. The filter plant is located directly across the Allegheny River from the Brilliant Pumping Station, from which most of the water used in the city is pumped. It is true that it was necessary to build conduits under the river to bring the filtered water to the pumping station, but the case would have been the same if rapid filters had been built instead, for, if the speaker's memory is correct, the best site on which filters of either type could have been built was that on which the slow sand filters were constructed.

For the four slow sand filter plants in Philadelphia mentioned by the author, namely, Torresdale, Upper Roxborough, Lower Roxborough and Belmont, and of which the speaker had charge of the design, no trouble was experienced in securing suitable sites.

As regards the Lower and Upper Roxborough filter plants, local conditions were of the greatest importance in that it was necessary to utilize existing reservoirs as settling basins, as well as existing pumping stations and pipe lines. Somewhat similar conditions existed with reference to the Belmont plant, although in this case there was no reservoir which could be utilized as a settling basin.

The Torresdale slow sand filter plant is the largest single plant in the world, and if difficulty had been experienced in securing suitable sites for such works it would naturally be expected to have occurred here. Such was not the case, however, and the plant was built inside the city limits. It is true that the plant was located some distance upstream from the closely built-up part of the city, and that a conduit about 2½ miles in length was constructed through which filtered water is delivered to the Lardner's Point pumping station, but it was good judgment on the part of the city authorities to locate the works where they did.

The Delaware River at Philadelphia is tidal and is polluted by the discharge of sewage from the city, and it was to avoid pollution as well as to secure a site that the plant was built as far upstream as it was. It might be added further that the city has already constructed a small sewage disposal works just below the Torresdale filter plant (the Pennypack Creek works), where sewage is treated and the effluent disinfected before discharge into the river, in order to guard against raw sewage from the nearest point from reaching the intake of the Torresdale filters. Had rapid sand instead of slow sand filters been built at Torresdale there is no question in the speaker's mind but that such a plant would have been built as far upstream as were the slow sand filters.

One might fancy, possibly, that trouble would be experienced by the city of New York in securing sites for filters. Such is not the case. Over ten years ago the Commission on Additional Water Supply, in its investigations for an additional water supply for New York City, looked into the question of filter sites not only for the additional supply but also for the Croton supply. Sites were found where slow sand filters could be built in close proximity to the existing Croton Aqueduct, and sites for slow sand filters were also found and surveyed where the additional supply could be filtered along the line of the proposed new aqueduct. Some years later the Department of Water Supply actually prepared detailed plans for a slow sand filter plant for the Croton supply, on a site located in New York City, namely, in the east basin of Jerome Park Reservoir, the construction of which was suspended pending a decision as to the filtering of the supply. In the case of New York City it is not a question of going a long distance and building an expensive conduit to get a filter site, but rather a question of going a long distance and building an expensive conduit to get water.

At Cincinnati, Ohio, the rapid sand filter plant, which is the largest of its kind in operation in the world, was built well upstream above the city where plenty of land was available for either rapid or slow sand filters. The filtered water is discharged through a long conduit to the main pumping station from which it is pumped to the reservoirs in Eden Park. This plant was built for the future as well as for the present and it was good judgment to locate the plant as far upstream as it was.

At Columbus, Ohio, the rapid sand filters were built at some little distance upstream above the city. It was a case here of going upstream to get out of the flood zone, although sufficient land for either rapid or slow sand filters was available nearer the city. At the point where the plant was finally built there was sufficient land for either rapid or slow sand filters. In acquiring the land for this particular plant it was found that it would be about as cheap to acquire the whole tract of land on which the works were built as to acquire only so much of the same as would be needed for the plant alone.

Perhaps enough has been said to point out that factors other than the area of land alone have to be taken into account, not only for slow but for rapid sand filters as well, in selecting a suitable site.

Cost of Construction.—The author also considers, in Table 1, the cost of construction of different types of filters. It is exceedingly difficult to compare satisfactorily the costs of construction of different plants, even where the fullest information regarding the same is available. Those who are not well posted as to the history of some of the plants cited in the table may possibly be misled as to the cost of building both slow and rapid sand filters if they accept the figures of the author without full knowledge of local conditions.

One of the features which very materially affects the cost of such works is the total reservoir capacity provided, that is, the combined capacity of the settling basins and of the clear water reservoirs. To illustrate: The rapid sand filter plant at Little Falls, N. J., which, in the author's table is the most expensive one cited, and which cost $15,000 per 1,000,000 gals. daily capacity, has a coagulating basin capacity of 1.3 hours and a filtered water reservoir capacity of 2.6 hours, or 3.9 hours total reservoir capacity. At Columbus, Ohio, the rapid sand filter plant there, which the author states cost $13,000 per 1,000,000 gals. daily capacity, the next to the highest in cost cited, has a settling basin capacity of 12 hours and a filtered water reservoir capacity of 8 hours, making a total reservoir capacity of 20 hours, or five times as much reservoir capacity as that of the Little Falls plant. If the reservoir capacity of the Little Falls plant had been approximately that of the Columbus plant the cost of construction of the Little Falls plant would have been materially increased over that given by the author. Again, the New Orleans rapid sand filter plant might be cited, which has 35.2 hours total reservoir capacity, or practically nine times as much reservoir capacity as that of the Little Falls plant. Other factors which affect the cost of construction are the character of the raw water, the rate of filtration, the character of the construction of the works, etc.

In his reference to the Albany slow sand filter plant the author gives its capacity as 20,000,000 gals. daily. The Albany plant as originally built before the pre-filters were added had a capacity of 15,000,000 gals. daily. The addition of the pre-filters increased the capacity of the plant very materially so that at the present time the capacity is probably in the neighborhood of 28,000,000 gals. daily. If the capacity is taken at 28,000,000 instead of 20,000,000 gals. daily the cost of the plant would be about $14,300 instead of $20,000 per 1,000,000 gals. daily capacity as given by the author.

The Philadelphia slow sand filter plants were expensive plants to build. They differ in one way from many of the other filters of the same type that have been built in that underneath the filter floors and carried up all around the sides of the filter is a layer of puddle. This item alone materially increased the cost of construction. The Lower Roxborough and Upper Roxborough plants were built on high ground in an isolated section several miles from the nearest railroad, and the cost of delivering materials to such plants was higher than would ordinarily be the case.

In the cost of the Lower Roxborough plant the author did not include the cost of the Lower Roxborough Reservoir which was built many years before, and which supplies settled water to the filter plant. Again, a similar condition exists at the Upper Roxborough filter plant with regard to the settling basin. The New Roxborough Reservoir was built some ten years earlier than the filter plant, and the author has not included its cost in the cost of the filter plant. Strictly speaking, the costs of the reservoirs should be included in the costs of these two plants so that the figures would be comparable with the costs of the other slow sand filters cited.

The Philadelphia plants were built during a period of very high prices, and to use the costs of construction of these plants to indicate the reasonable cost of slow sand filters may be very misleading except to those who are familiar with the early history of these works and who are aware that the costs were high and that the plants could be duplicated at less cost.

The largest slow sand filter plant under construction in America at the present time is at Montreal, and, when completed next year, will have a capacity of 60,000,000 U. S. gals. daily capacity. The total cost of the plant, on the basis of the lump sum contract prices, including the low lift pumping station, will be about $22,600 per 1,000,000 gals. daily capacity. Deducting the low lift pumping station the cost will probably be about $21,000 per 1,000,000 gals. daily capacity.

It would have been interesting if the author had cited the cost of the slow sand filter plant which was completed at Toronto about two years ago. This plant has a capacity of 48,000,000 U. S. gals. daily, assuming one-sixth of the filter area to be held in reserve, and based on a rate of filtration of 6,000,000 U. S. gals. per acre daily, the rate for which the plant was designed. The cost of the plant, omitting the low lift pumping station, was only about $12,700 per 1,000,000 gals. daily capacity.

In considering the weighted average cost of slow sand filters given by the author, namely, $32,600 per 1,000,000 gals. daily capacity, it may be well to bear in mind that the Montreal plant will cost only about $21,000, that the Albany plant cost about $11,300, and the Toronto plant only $12,700 per 1,000,000 gals. daily capacity.

In referring to the cost of rapid sand filter plants the author cites the Columbus plant as costing $13,000 per 1,000,000 gals. daily capacity. This plant was designed and built under the speaker's direction and is a water-softening as well as a rapid sand filter plant. The cost of this plant was given in great detail in Trans. Am. Soc. C. E., Vol. LXVII, 1910. (These cost data were published in ENGINEERING & CONTRACTING of Feb. 9, 1910.) The speaker is not informed as to what items the author included in arriving at the cost of the Columbus plant, but in the speaker's judgment the Columbus plant, considered as a rapid sand filter plant alone, cost nearer $15,000 than $13,000 per 1,000,000 gals. daily capacity, the figure given by the author.

Another rapid sand filter plant which the author might have cited is that at Toledo, Ohio. Part of the plant was built for a capacity of 60,000,000 gals. daily, although the present capacity of the works is considerably less. Including only such items as are chargeable to the filter plant proper the works cost about $14,500 per 1,000,000 gals. daily capacity.

Another rapid sand filter plant which might have been cited is that at Grand Rapids, Mich. The plant was completed inside of the last two years and has a capacity of 20,000,000 gals. daily. The cost of the plant, as given to the speaker by the Grand Rapids officials last year, including such items as are chargeable to the filter plant proper, was $16,300 per 1,000,000 gals. daily capacity.

In December, 1912, the city of New York received bids for a rapid sand filter plant to be located at Jerome Park Reservoir and having a capacity of 320,000,000 gals. daily. The speaker is more or less familiar with the plans for the proposed Jerome Park filters as he served as one of a Commission of Engineers appointed by the Board of Estimate of New York to report on the same. Taking the lowest bid received and adding to it the cost of the buildings and other necessary work, the Jerome Park filter plant, which would have been the largest rapid sand filter plant in the world, would have cost about $18,400 per 1,000,000 gals. daily capacity. When the plant is built, and it is greatly to be hoped it will be built soon, the actual cost will probably be in the neighborhood of $20,000 per 1,000,000 gals. daily capacity, as much of the excavation for the plant has already been completed.

The author gives the cost of the Cincinnati rapid sand filter plant, which has a daily capacity of 112,000,000 gals., as $11,400 per 1,000,000 gals. daily capacity, and states that the cost of the large, plain sedimentation basins is not included. At Cincinnati there are two large settling basins to which the raw water from the Ohio River is pumped. The water is first settled in these two basins, and is then delivered to the coagulating basins at the filter plant. There is no question in the speaker's mind but that the settling basins are part of the filter plant at Cincinnati, but just how much of the cost of the same should be chargeable to the filter plant may be a question. Mr. J. W. Ellms, the superintendent in charge of the filters at Cincinnati, in a paper printed in the Journal of the Association of Engineering Societies in January, 1912, states:

> The settling reservoirs, which have a capacity of 330,000,000 gals. of available water, are in part a portion of the water purification plant, although they also serve the purpose of storage basins and were designed for such a use quite as much as they were for sedimentation purposes.

The two settling basins cost $1,521,000, or about $13,600 per 1,000,000 gals. daily capacity of filter plant. Adding this cost to that of the filter plant would give a total cost of $25,000 per 1,000,000 gals. daily capacity. As the settling basins serve as storage reservoirs also it may be reasonable to charge the filter plant with perhaps only half their cost. On this assumption the cost of the settling reservoirs chargeable to the filter plant would be $6,800 per 1,000,000 gals. daily capacity, thus making the total cost of the filter plant $18,200 per 1,000,000 gals. daily capacity.

Still another plant which the author might have cited, and among the best in the country, is that at New Orleans, which has a capacity of 40,000,000 gals. daily. Including only such items as are chargeable to the filter plant proper the cost of the New Orleans plant was about $30,200 per 1,000,000 gals. daily capacity.

The weighted average cost of the Columbus, Toledo, Grand Rapids, Cincinnati and New Orleans rapid sand filter plants is $18,600 per 1,000,000 gals. daily capacity, while the author gives a weighted average cost for rapid sand filters as $12,100. In other words, the weighted average cost of the five plants just cited, all of which are in operation and which are among the best in the country, is over 50 per cent higher than the weighted average cost given by the author.

The speaker has but little further to say on the subject of costs except that, in his judgment, the weighted average costs as given by the author are too high for slow sand filters and are too low for rapid sand filters. Similarly the fixed charges on the costs of construction would respectively be too high for slow sand and too low for rapid sand filters.

The speaker is not presenting any brief for slow sand filters. The rapid sand filter is more flexible than the slow sand filter and in the majority of cases in the United States is better adapted to the purification of water than is the slow sand filter. The slow sand filter has done and is still doing good work in this country, and the present status of water purification is, to a large extent, due to the introduction of the slow sand filter.

Law and Rules Regarding the Inspection of Dams and Reservoirs by the Board of Civil Engineers of the State of Connecticut.

The present law in Connecticut on the state censorship of the design and construction of dams was enacted on June 7, 1913. It is entitled, An Act Concerning the Inspection of Reservoirs by State Engineers. A digest of the law follows:

The commissioner of rivers, harbors and bridges, who is a civil engineer, and one civil engineer living in each congressional district appointed by him, constitutes the board of civil engineers. This board has supervision of all the dams and reservoirs of the state which in failing could endanger life or property. The members hold office for terms of two years. They are an organized body with officers and all their official acts are recorded. Each member is paid an annual salary of $100 and, in addition, $10 per day when actually employed. All their necessary and reasonable expenses are paid.

Upon the application in writing of two or more persons or corporations who would suffer loss or damage by the failure of any dam or reservoir, a member of the board of civil engineers promptly inspects the structure and if in his opinion it is not sufficiently strong to withstand the action of water under any circumstances which may reasonably be expected to occur, he at once serves notice on the person or corporation owning or having the care and control of the structure to place it in a safe condition under his supervision. When such repairs are completed and accepted by the engineer he issues a certificate of acceptance to the person or corporation, who records it in the land records of the town in which the dam or reservoir is located. If the engineer finds the dam or reservoir to be safe the expense of inspection is paid by the applicants.

Before any person or corporation constructs a dam or reservoir or alters an existing one in a locality where life or property may be endangered through insufficient strength of structure, the plans, specifications and necessary data for such dam or reservoir are submitted to a member of the board, who examines the ground where the dam or reservoir is to be built and the plans and specifications therefor. If he approves these he issues a certificate authorizing the construction of a new dam or reservoir. If the state engineer deems it necessary he may, before issuing the certificate, call in for consultation and advice the state board of civil engineers or any member of the board.

The engineer under whose authority a dam or reservoir is being constructed or a dam is being altered determines whether the structure will be safe, and when the work has been completed to his satisfaction he issues a certificate approving it. This certificate is recorded in the land records of the town in which the dam or reservoir is located.

Any person aggrieved by any decision of a member of said board of civil engineers is given a hearing by the board. Any decision of the board may be appealed from to any judge of the superior court within 20 days of the date of such decision.

Every person or corporation who shall build or repair any dam or reservoir except in compliance with the provisions of this act, or shall use any dam or reservoir when constructed or repaired without obtaining the certificate provided for in this act, shall forfeit $500 for the use of the state. Any person or corporation constructing or repairing a dam or reservoir, or using any such dam or reservoir when constructed and repaired without complying with the provisions of this act, may be enjoined from constructing or using such dam or reservoir.

The state's attorney of the county in which the dam or reservoir is located, upon complaint of any member of the board, institutes an action to recover this forfeiture and to enjoin the construction and use of the dam or reservoir.

The board of civil engineers recently issued a small pamphlet containing a copy of the act presented above in substance and also containing the set of rules adopted by the board. Following the statement of the rules the pamphlet gives certain information for the guidance of those to whom the act applies. The chief provisions of the rules are quoted herewith and the information mentioned is presented in substance:

RULES ADOPTED BY BOARD OF CIVIL ENGINEERS.

1. There shall be submitted for the preliminary approval two copies each of drawings, specifications, description of proposed work, etc. If approved, one copy of each shall be so marked and returned to the owner, the other copy being retained by the Board member for use during the construction period.
2. Before obtaining the final certificate of approval, the owner shall, for permanent record, file with the member of the Board issuing the certificate, plans on tracing cloth or plans on cloth by the in-ko-graf or other equally permanent and satisfactory process of printing, and specifications or written description which show and describe the work as actually constructed. These plans and written data shall be certified as correct by a person who was in responsible charge during construction and who knows to his best knowledge and belief that the facts certified to are correct.
3. The permanent plans and records shall, from time to time, or at the end of each year, be filed with the Librarian of the State Library by the member of the Board who issues the final certificate of approval.
4. Certificates shall be issued from the certificate book belonging to the congressional district in which all or the principal part of the dam or reservoir is located.
5. Members of the Board, while acting in their official capacity, shall not render professional services to owners in connection with the dams and reservoirs concerning which they are officially engaged.
6. A member of the Board who may be employed as designing or constructing engineer of a dam shall, by reason of such employment, be disqualified to act in an official capacity as inspector of such dam, or to issue certificates of approval therefor.

INFORMATION.

The owner should furnish to the member of his district plans and specifications in sufficient detail to enable him to understand just what kind of work is contemplated. The act mentions that necessary data are to be supplied to the board member. This is information such as the area of watershed, location of dam, the character of the soil or rock that the dam will rest on, whether or not there are any other dams upstream imposing a hazard on the dam in question, or other data necessary to be considered by the member in order to determine if the proposed construction will in his judgment be safe. Owners are urged to send full information with the plans and specifications when making application for preliminary approval in order to avoid delays otherwise necessary to obtain such information.

Rule 2 of the board requires plans on tracing cloth or plans on cloth by the in-ko-grai or other equally permanent and satisfactory process of printing, showing the work as actually built, to be filed as the permanent record. Specifications or written descriptions are also to be filed. Blueprints, even when made on cloth, are not permanent, therefore are not accepted. The permanent records will be kept in the vault at state library, and it is thought will be of great value to both the owners of dams and the board of civil engineers if in the future it is desired to do work on a dam previously inspected. It has been the history of nearly all the dams built in the state that sooner or later the owners desire to raise, reconstruct or otherwise change them, and very frequently find they do not possess plans or descriptions of the old work that they know are correct. The board asks the co-operation of the owners of dams in making the permanent records complete.

There are six members of the board of civil engineers, one member at large and one from each of the five congressional districts of the state. It is customary to make application for approval to the member representing the congressional district in which the dam is located, but the law permits any member of the board to act in any location in the state.

Sewerage

Designing the New Intercepters of the Comprehensive Sewerage System for Cincinnati.

In our issue of June 3, 1914, we called attention to the exceptional value of the Cincinnati sewerage report as a series of carefully worked out and adequately reported sewerage engineering studies. In that issue we published an article on the procedure in designing new and relief storm water sewers. The present article gives similar information with reference to the design of the new intercepting sewers. The portion of the report from which the information here given is taken was prepared by Mr. E. J. Miner, who acknowledges the assistance of Messrs. B. F. Perry, F. C. Dugan and H. T. French, and the assistance of Mr. C. W. Sherman of the firm of Metcalf & Eddy in making the engineering investigations and in preparing the report on intercepting sewers, respectively.

GENERAL PROBLEM.

The present sewerage system consists of a large number of disconnected sewers, each serving a limited district. In some cases these districts are very small; in others, very large. Each sewer has been constructed to discharge into the nearest water course thought adequate to take away the sewage. Nearly all of the existing sewers are built on the combined system. Accordingly, during dry weather the sewers discharge domestic sewage into the streams and during storms, street washings and roof water, combined with the sewage.

In the case of districts adjoining the Ohio River this method of discharge has not in general proved objectionable. As a rule, the trunk sewers empty into the river below the water level, so that, except in the immediate vicinity of the outlet, the fact that sewage is being discharged into the river is hardly noticeable. In the case of the small streams flowing through the city, however, conditions are very different. During dry weather these streams carry very little water, and the result is that the sewage flow constitutes a sufficient percentage of the water flowing in the streams to be noticeable and offensive. Stagnant pools of putrifying sewage are not uncommon, and during the dry months these streams are not only a nuisance, but are a menace to the health of the community. At times of storm the accumulated sewage is flushed out and carried to the Ohio River.

With the exception of the sewers discharging into the Ohio River, practically all of the sewage of the city is discharged into Mill Creek and its tributaries and Duck Creek. These two streams receive at least 65 per cent of the sewage of the city, and are accordingly in very bad condition.

It is necessary to provide intercepting sewers to receive the sewage from the trunk sewers and to convey it to a suitable point of disposal. These intercepters were designed not only to receive sewage from the existing and future sewers, but to convey it to a point where it may be discharged without offense, or treated in such a way that the resulting effluent may be discharged into the streams.

There are at present a few sewers which are called intercepters and which serve to some extent as intercepting sewers; that is to say, they receive the dry weather discharge of one or more sewer districts and convey it to a point where the stream flow is assumed to be greater, so that the dilution of the sewage is more thorough. These sewers will serve as branches to the main intercepters which will be required in any comprehensive scheme.

Disposal by dilution will be sufficient for the present. It may be, however, that ultimately the discharge of untreated sewage into rivers and streams will be prohibited by law, and to fulfill sanitary requirements it will be necessary to treat all sewage before its discharge into streams. Accordingly the intercepters have been designed with a view to the ultimate collection of the sewage at suitable sites for such treatment as it may require.

COLLECTION OF SEWAGE.

Intercepting sewers following in a general way the course of the streams into which the trunk sewers now discharge will be necessary to collect the sewage from the several sewer districts. It is obvious that three main intercepters or systems of intercepters will be required: one in Mill Creek Valley, one in Duck Creek Valley and one following the bank of the Ohio River.

In such a case as the Mill Creek Valley it is sometimes practicable or desirable to construct two or even three intercepters. For instance, conditions might make it desirable to build high level intercepters at the edge of the high land on both sides of the stream, each of which would intercept the greater part of the sewage tributary to the stream from its side. The third intercepter, which might not be needed if the valley below the high land were not developed, would closely adjoin the stream and would receive the sewage from the districts lying below the high level intercepters. Such a project is particularly desirable where pumping the sewage is necessary, in order to collect as much of the sewage as possible at a high level and thus reduce the amount to be lifted or the height to which it must be pumped. This method was not found practicable for Cincinnati, and it is planned to construct a single main intercepter closely adjoining Mill Creek, a similar one near Duck Creek and a third along the bank of the Ohio River.

ECONOMIC PERIOD OF DESIGN.

In designing intercepting sewers it is obviously necessary to make provision for a reasonable amount of growth in order that they may not be soon outgrown. It is also necessary, first, that engineering works should not be constructed on too large a scale, so that they cannot operate efficiently under present conditions, and, second, that expense of construction for future requirements should not be so great that the burden of debt imposed at present becomes unreasonable. It may readily be seen that in some cases the saving of expense resulting from providing works sufficient for the present may be sufficient amply to provide for the additional construction that may be made necessary when larger works are needed.

The economic period for which provision should be made in the design of intercepting sewers depends on various conditions, among which the rate of growth of the city is one of the most important, and is a condition which cannot be absolutely fixed. In general it is desirable to provide for a period ranging from 30 to 50 years from the date of design. In the present study it has been concluded that approximately 40 years is a suitable period for which to make provision and, rounding off the figures, designs have been made on the basis of the requirements of the year 1950.

AREA.

From 1802 to 1848 the area of Cincinnati was three square miles. Various small areas were annexed from time to time until, in 1910, the total area was approximately 50 square miles. During 1911 an area of over 16 square miles was annexed to the city and in 1912 the area annexed was about 3.5 square miles, bringing the total area of the city up to about 70 square miles.

On Jan. 24, 1913, a meeting was held at which eleven representatives of local real estate firms were present. The subject of the future growth of the city in area and in population was discussed at considerable length and the views of all present were freely expressed. After giving due consideration to the opinions expressed by the real estate experts, and also to the past growth of the city both in area and population, and to local conditions in general, it has been decided to base the design of the comprehensive system of sewers upon an estimated future area of about 140 square miles.

POPULATION.

There are doubtless many persons in Cincinnati who are sufficiently optimistic to believe that the city will have a population of 1,000,000 within the next 40 or 50 years. The history of Cincinnati during the past 60 years, however, has little in it to warrant such an estimate. On the other hand, it hardly seems wise to base the design for a system of sewers which should be adequate for the city for a period of nearly 40 years on a rate of increase in population which is not greater than that which has been enjoyed by the city in the past.

Cincinnati has had an exceptionally slow rate of growth, but the natural advantages, its location and the enetrprise of its citizens would seem to warrant the opinion that a more rapid growth will occur in the future. Careful consideration of the data which has been accumulated in connection with this study, and of the local conditions, leads to the conclusion that a comprehensive system of sewers designed at the present time for a period of about 40 years in the future should be based upon a future population of about 720,000 persons and a rate of growth averaging from 20 per cent per decade in the immediate future to 17 per cent per decade 40 years hence. The population of the city at the end of each decade, should this estimate prove correct, will be: 1920, 440,000; 1930, 520,000; 1940, 615,000; 1950, 720,000.

By basing the designs for a sewerage system upon these estimates provision is made for a future population approximately twice as great as that of Cincinnati at the present time.

DISTRIBUTION OF POPULATION.

The area and population of the city of Cincinnati and the number of persons per acre at the end of each decade from 1850 to 1910, inclusive, and in 1912, and as estimated or assumed at the end of each decade from 1920 to 1950, inclusive, are given in Table I.

As a basis for computing the quantity of sewage to be provided for by the intercepting sewers, the estimated future population was distributed among the several sewer districts.

The boundaries of each voting precinct and of the several sewer districts were plotted upon a map, scale of 1,000 ft. to 1 in. By combining the populations in the several precincts and parts of precincts within a given sewer district the total population now residing in each sewer district was determined. In determining the density of population in the various districts allowance was made for unpopulated areas, as parks, cemeteries and railroad yards, by deducting these areas from the area of the district.

TABLE I.—DENSITY OF POPULATION, CITY OF CINCINNATI, 1850-1950.

Year.	Population.	Area, sq. miles.	Density persons per acre.
1850	115,435	6.16	29.3
1860	161,044	6.93	36.3
1870	216,239	19.05	17.7
1880	255,139	23.53	16.9
1890	296,908	23.73	19.6
1900	325,902	35.27	14.5
1910	364,463	50.26	11.3
1912	380,305	69.85	8.5
1920	440,000	82.45	8.3
1930	520,000	99.30	8.2
1940	615,000	119.35	8.1
1950	720,000	141.50	8.0

The areas of the sewer districts vary greatly, the small districts for the most part being those which are directly tributary to the Ohio River and in the older portion of the city. The area of the smallest district is 10.5 acres, while that of the largest well-sewered district is 4,068.7 acres. The present density of population varies from 0.2 person per acre to 104 persons per acre. The estimated future density of population varies from 0.4 person per acre to 110 persons per acre. This is considered to be a liberal basis for the design of the intercepting sewers, as the deficiencies of some districts will offset the excesses of others; but where individual sewer systems are to be designed a more liberal allowance for future growth should be made, because no such compensating conditions can be relied upon for the relatively small sewer districts.

Main Drainage Districts.—The city may be naturally divided into three main drainage districts, that served by Mill Creek, that draining directly into the Ohio River, and that served by Duck Creek; to which should be added two small areas within the estimated future city limits, one of which is naturally drained by Rapid Run and Muddy Creek, and the other by Sycamore Creek. It appears that provision has been made for an estimated future population of over 300,000 persons in Mill Creek Valley and approximately an equal number in the Ohio River District, and that the estimated future population in the Duck Creek Valley is nearly 100,000 persons.

PROPORTION OF FUTURE AREA OF CITY ASSUMED TO BE DEVOTED TO RESIDENTIAL, INDUSTRIAL, COMMERCIAL AND PARK PURPOSES.

The quantity of sewage contributed from residential areas is much less than that from industrial and commercial areas. To arrive at a reasonable basis of design, a study was made to determine the portions of the city which in the future will be devoted to residential, industrial and commercial purposes, and to parks and cemeteries. From the last two the quantity of sewage for which provision should be made is very small, being made up chiefly of ground water. As hereinbefore stated, it has been assumed that the area of the city in 1950 will be approximately 140 square miles. With the aid of the opinions expressed by representatives of local real estate firms an estimate has been made of the probable limits of the industrial and commercial areas. The distribution of the area within the assumed limits of Cincinnati as of 1950 is as follows:

Area.	Square miles.	Percentage.
Industrial	12.79	9.04
Commercial	1.13	0.80
Residential	123.30	87.11
Cemetery and park	4.02	2.83
Railroad yards	0.31	0.22
Total	141.55	100.00

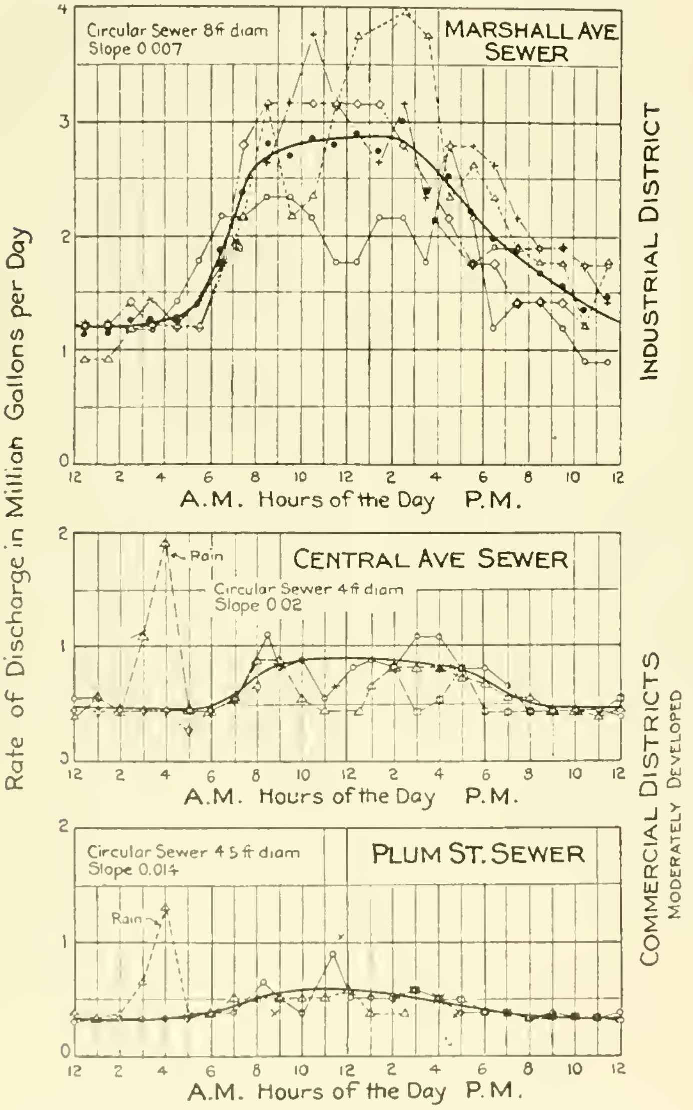


Fig. 1. Hourly Variation in Rate of Sewage Flow in Typical Commercial and Industrial Districts, Cincinnati.

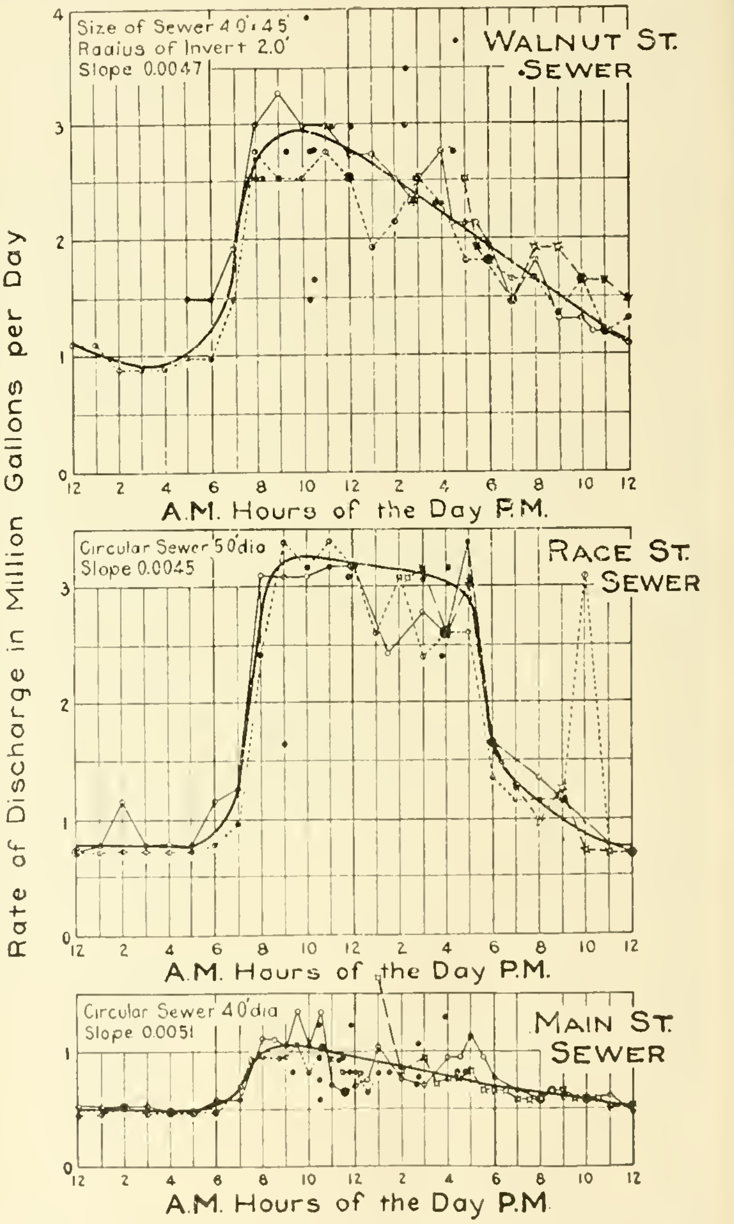


Fig. 2. Hourly Variation In Rate of Sewage Flow In Highly Developed Commercial Districts of Cincinnati.

QUANTITY OF SEWAGE TO BE HANDLED BY THE INTERCEPTING SEWERS.

Capacity Required.—The system of intercepting sewers was designed to receive and convey to a point of discharge all of the sewage contributed in 1950 by the several district sewers with which they connect, at times when storm water is not actually running into the city sewers. It is therefore important that the intercepters be large enough to carry the maximum quantity of sewage collected by the trunk sewers. This quantity will vary from hour to hour, from day to day and from season to season. In the spring, when the ground water is high and relatively large quantities of it seep into the sewers, the quantity collected will be large and the intercepters should have sufficient capacity to convey it to the point of discharge. It is not intended, however, to provide capacity in the intercepters for any substantial quantity of actual storm water, the plan being to provide overflows from the trunk sewers, so that when the capacity of the intercepters is reached the excess flow may discharge directly into the neighboring watercourses. This phase of the design will be covered in a succeeding article.

When first constructed the intercepters will have a surplus capacity, intended to provide for the expected increase in area and population of the city, which may be utilized to carry small quantities of storm water so that overflow from the trunk sewers will not take place quite as frequently as in later years, when the normal flow of sewage has reached approximately that for which the intercepters have been designed. The quantity of sewage collected varies greatly from hour to hour, as shown by Figs. 1 to 4. It also varies materially from day to day during the week, the smallest quantity usually being produced on Sundays and holidays and the maximum quantity as a rule on Mondays. The quantity of sewage from commercial districts is greatest during business hours, as indicated by Figs. 2 and 4, which show the hourly variations in the flow in several sewers.

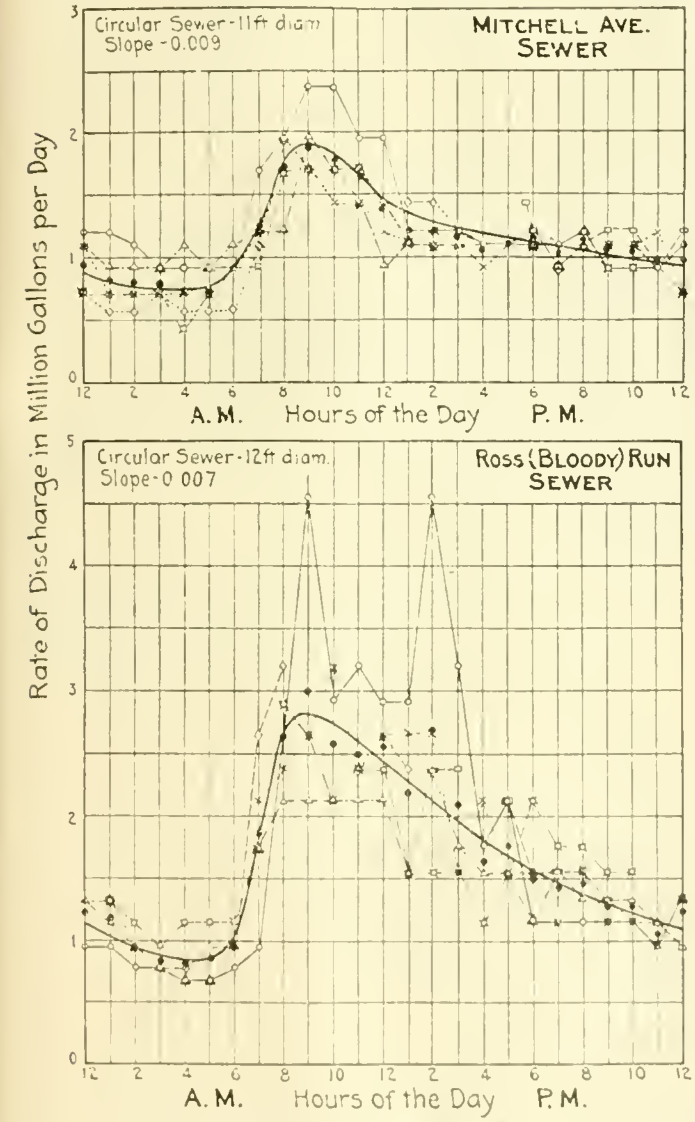


Fig. 3. Hourly Variation in Rate of Sewage Flow in Typical Residential Districts of Cincinnati.

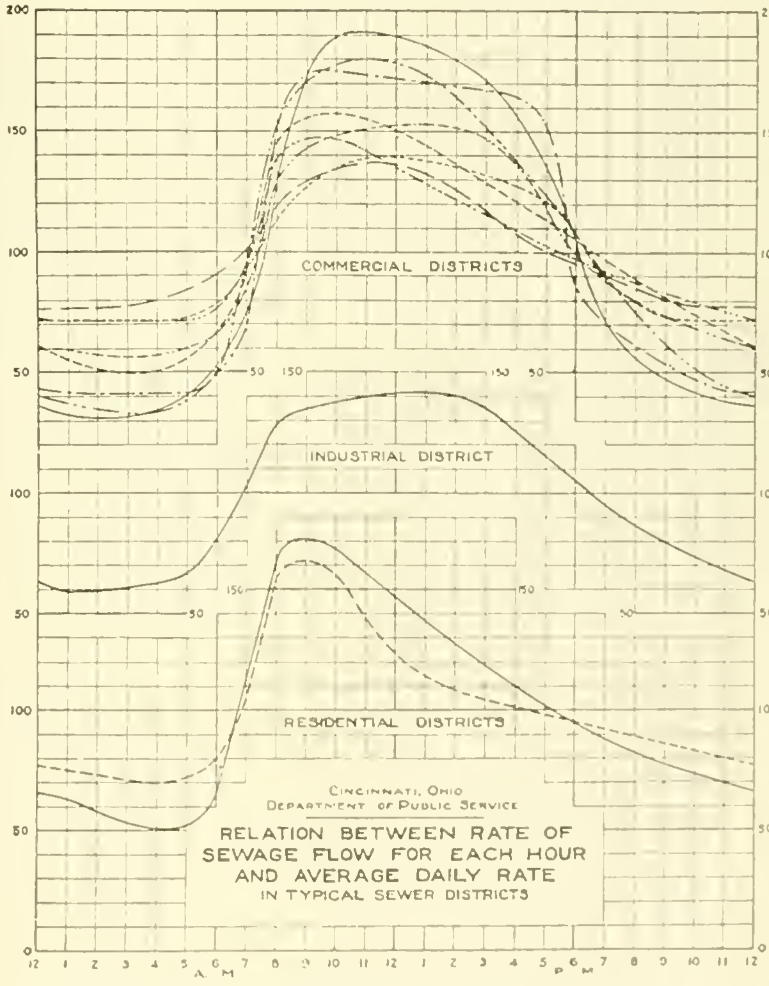


Fig. 4. Curves Showing the Relation Between the Rate of Sewage Flow for Each Hour and Average Daily Rate in Typical Sewer Districts of Cincinnati.

The quantity of industrial waste fluctuates, depending upon the character of the industry, the condition of business and the hours of work. Paper mills are usually operated throughout the 24 hours, so that the rate of discharge of their wastes is generally fairly uniform from hour to hour. Packing houses, tanneries and woolen mills, on the other hand, are usually operated from 7 a. m. to 6 p. m., except for one hour at noon, and the discharge from such plants is therefore largely confined to those hours. In some industries, however, the processes employed are such that the wastes are discharged by the emptying of vats or otherwise, so that the discharge takes place intermittently, with corresponding fluctuations in the rate of flow.

The quantity of ground water which finds its way into the sewers varies in a general way with the variations in rainfall. Usually a hard and long-continued rain is followed by a material increase in the flow of sewage, due to infiltration of ground water. In the spring when the frost is coming out of the ground, leaving it more porous than at other seasons of the year, considerable surface water finds its way into the earth, so that the rate of infiltration into the sewer system is then usually much higher than during other seasons. The least quantity of ground water seeps into the sewers during seasons of extreme drought, or during the winter when the ground is securely sealed by frost.

In order that the intercepting sewers may perform their functions it is necessary that they shall have sufficient capacity to carry the sewage, not only on the day and at the hour when the normal flow of sewage is greatest, but also at times when the discharge of wastes from the commercial districts is highest, when the discharge from the industrial districts is a maximum, and also in the spring of the year when the infiltration of ground water is greatest. These maximum quantities may all occur at the same time, so that they must all be taken into consideration in determining the size of the sewers. At the same time it is important not to lose sight of the fact that the maximum rate of flow in the intercepter at any point is not the sum of the maximum rates of discharge of all the contributing trunk sewers.

Ordinarily the maximum discharge from trunk sewers occurs between the hours of 10 a. m. and 2 p. m. If the intercepter is so long that several hours are required for sewage discharged into it at its upper end to reach the point of discharge of the last trunk sewer, near its lower end, the maximum discharge from the lower sewer will have left the intercepter before the maximum discharge from the upper sewer has reached the lower point. In other words, there is a tendency for the peak flows in the several contributing trunk sewers to compensate each other, and on this account the rate of flow in an intercepting sewer is always more uniform than the rates of flow in the contributing trunk sewers. For example, the maximum rate of flow in the Vine Street Sewer on Tuesday, Nov. 5, 1912, was 179 per cent of the average for the 24 hours, whereas it is to be expected that the maximum rate of flow in the Ohio River Intercepter, into which the Vine Street Sewer will discharge, will not ordinarily exceed 135 per cent of the average flow for 24 hours.

Quantity of Sewage from Residential Areas.—The quantity of sewage collected by a system of sewers serving a strictly residential district depends upon the quantity of water used and discharged into the sewers and upon the amount of ground water seeping into them. It is therefore important to take into consideration the consumption of water, as well as to measure the flow of sewage in trunk sewers serving large residential areas in which there are no manufacturing and substantially no commercial districts.

The flow in two such trunk sewers, the Ross Run and the Mitchell Avenue sewers, has been measured. It was not practicable to place weirs in either of them, and it was therefore necessary to make measurements of the depth at frequent intervals of time and to compute the quantity from these measurements. These computations have been made by Kutter's formula with n=0.015.

All of the measurements were tabulated and for each sewer the several observations made at the same hour of each day were averaged and smooth curves, Fig. 3, were drawn to represent these averages. It is believed that these conventional curves represent with reasonable accuracy the rates of flow which may be expected at any hour of the day, between Monday noon and Saturday night, of any week during which the ground water is low and no storm water is reaching the sewers. Similar diagrams in which the rates of flow are shown in terms of per cent of the average rate for the twenty-four hours were made from the conventional curves, and are included in Fig. 4.

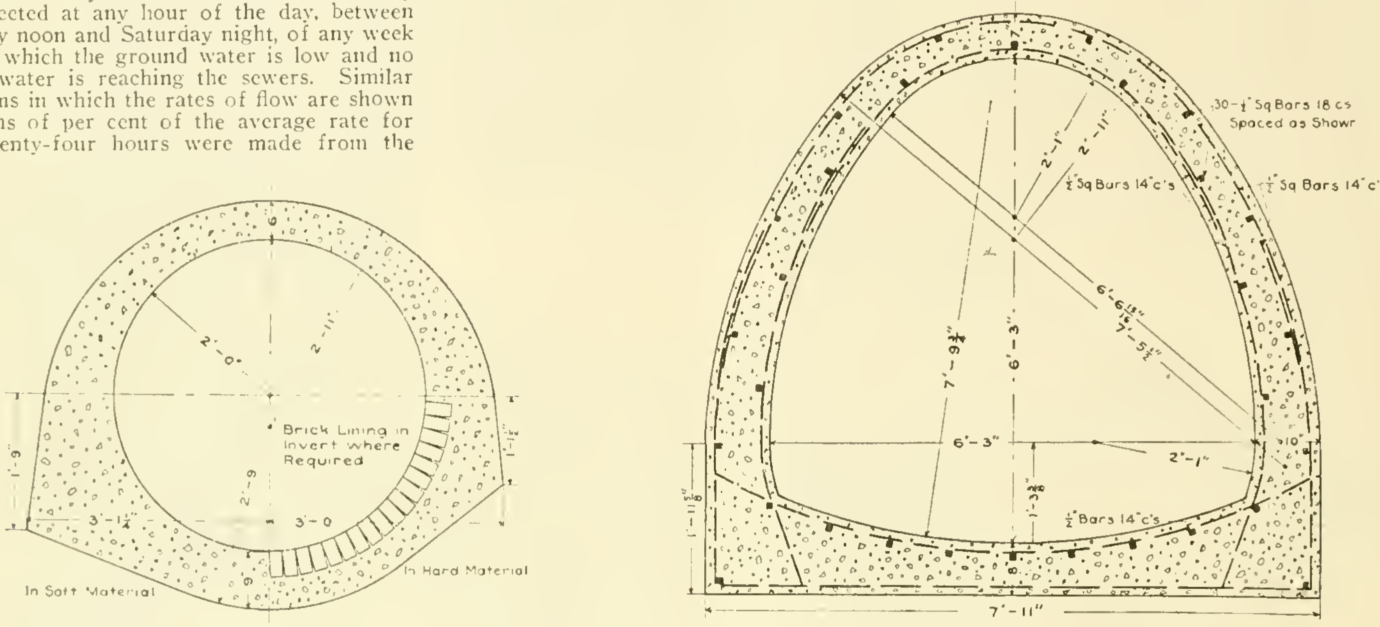


Fig. 5. Typical Circular and Semi-Elliptical Sections of New Intercepting Sewers, Cincinnati, Ohio.

The measurements of the quantity of sewage produced by the two residential areas showed averages of 92 and 80 gals. per capita per day, respectively, while the maximum rates were 157 and 128 gals. per capita, respectively.

Water Supply.—Very complete records of the quantity of water consumed have been kept by the Water Department. A table was prepared showing the total quantity of water consumed, the quantity of water per capita, the number of services, the number of meters in use and the per cent of services metered, together with the census, or estimated population served for the years specified. In 1912 17.6 per cent of the water services were metered.

From the data so compiled it appears that in the last 23 years the minimum daily rate of water consumption was 114.5 gals. per capita in 1890, that the maximum rate was 154.01 gals. per capita in 1895; and the average daily rate for the last 23 years was 130.6 gals. per capita.

During recent years an effort has been made to increase the proportion of services metered so as to check the waste of water, the experience of other cities proving that by the use of meters the quantity of water consumed can be held down to reasonable figures. In Cincinnati the consumption has been considerably higher than in a number of other large cities, but while the tendency of the times is toward a greater per capita consumption, it is expected that if a large proportion of the services are metered the daily rate of consumption will not materially exceed that of the present time, which may be fairly taken to be 125 gals. per capita. It is possible that this rate might be somewhat reduced, possibly to as low a rate as 100 gals. per capita daily. Taking into consideration, however, the tendency towards greater consumption, it does not seem wise to plan for a period of 40 years in the future without allowing for some increase. In these studies it has therefore been assumed that the daily consumption will ultimately, and within the period for which the intercepting sewers are designed, reach 150 gals. per capita.

It is commonly thought that practically all of the water supply eventually reaches the sewers. This, however, is a fallacy, for a very considerable part, probably not less than 40 per cent, is disposed of in other ways. It is evident that the water used for steam purposes, much of that used for manufacturing and mechanical purposes, that used by street and lawn sprinklers and the leakage from mains and services, as well as that used by water consumers not connected to the sewers, fails to reach the sewers.

An estimate of the quantity of water used for these several purposes in Milwaukee was made by the Milwaukee Sewage Disposal Commission in 1911 as follows:

Purposes.	Gals. per capita per day.	Per cent of total consumption.
Steam railroads	5	4.76
Manufacturing and mechanical	5	4.76
Street sprinklers	5	4.76
Lawn sprinklers	2½	2.38
Consumers not connected to sewers	7½	7.14
Leakage from mains and services	15	14.28
Totals	40	38.08

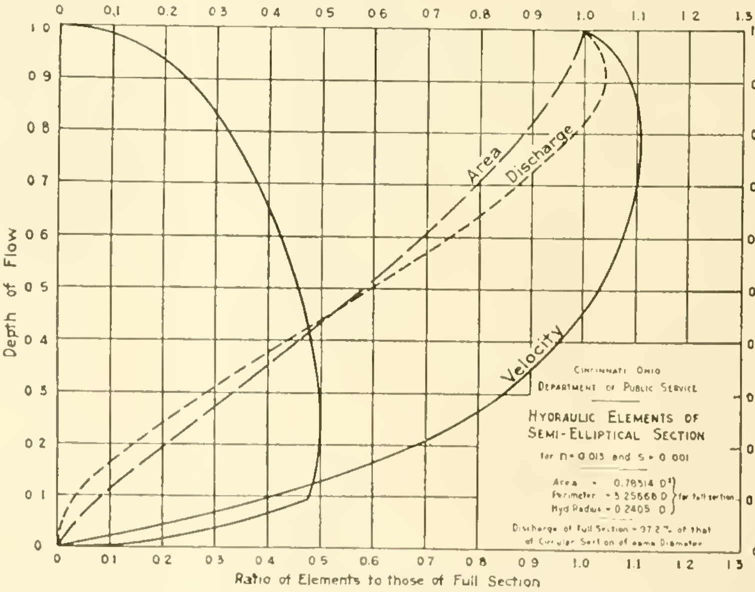


Fig. 6. Diagram of Hydraulic Elements Involved in the Calculations of Flow in the Semi-Elliptical Section of Intercepting Sewers, Cincinnati, Ohio.

The percentage of the water used for these purposes in Milwaukee was based upon a total average daily consumption of 105 gals. per capita. Applying the same percentage in Cincinnati, it would appear that the quantities of water given in Table II would be used for the several purposes indicated in the years 1912 and 1950, respectively.

With an assumed daily use of water of about 150 gals. per capita, 56 gals. of which do not reach the sewers, it has seemed reasonable to take the average quantity of domestic sewage, exclusive of ground water, at 90 gals. per capita per day and the maximum rate at which it will reach the intercepter at 135 gals. per capita per day. These estimates seem to be justified by the determinations of the daily flow of sewage in the Mitchell Avenue and Ross Run sewers, which were 80 and 92 gals., respectively, with maximum rates of 128 and 157 gals., respectively, per capita.

Quantity of Sewage from Commercial Districts.—The commercial district in Cincinnati is unusually well defined, and it is served by a number of parallel sewers, each draining a relatively small district, it was deemed wise to make measurements of the flow so that an intelligent estimate might be made of the quantity of sewage which will have to be taken into the intercepting sewers.

Accordingly, measurements were made of the flow in the sewers in Sycamore, Main, Walnut, Vine, Race, Elm and Plum streets and Central Avenue. Each of these sewers is between 3,000 and 3,500 ft. in length and serves an area of approximately 35 acres. It should be noted that the Plum Street and Central Avenue sections, although in the commercial district, have large resident populations. The variations in the rate of flow in these sewers are shown on Figs. 2, 3 and 4.

The measurements of the rate of flow of the sewers serving commercial districts were made during the extremely dry season, and it has been assumed that the infiltration of ground water at the time the measurements were made was negligible.

Of the total area of 227.2 acres in the commercial sewer districts gaged, the Main, Sycamore, Walnut, Vine, Race and Elm Street districts, with a combined area of 169.3 acres, may be considered as highly developed. In these districts will be found hotels, high-class office buildings, department stores and public buildings, with relatively few apartment houses. The streets are paved with asphalt and granite blocks. The Plum Street and Central Avenue districts, comprising an area of 57.9 acres, may be classed as moderately developed. In these districts will be found smaller hotels, smaller stores, wholesale houses and second-class business blocks, with a considerable proportion of tenement houses. A further area of 133 acres is considered as a semi-commercial area lying around that moderately developed.

The estimation of maximum rate of sewage flow from commercial areas (in addition to sewage contributed by the resident population, and not including ground water) has been made as follows:

The average of the maximum rate of flow for the six "highly developed" districts above mentioned is 81,508 gals. per acre daily (maximum points taken from conventional curves), which may be taken to fairly represent the maximum flow of sewage per acre from these districts if combined. The area is 169.3 acres, so that the maximum rate of discharge would be 13,800,000 gals. per day. The resident population is 4,744, and the maximum rate of flow of domestic sewage corresponding to this population at 135 gals. per day per capita would be 640,000 gals. per day. The difference, 13,160,000 gals. per day, is taken to represent the maximum rate of sewage flow due solely to the commercial development of the district, equivalent to 77,700 gals. per acre daily.

TABLE II.—ESTIMATED QUANTITIES OF WATER USED, BUT WHICH DO NOT REACH SEWERS, CINCINNATI, OHIO.

Purposes.	Gals. per capita per day based on total consumption of 125 gals., 1912.	Gals. per capita per day based on total consumption of 150 gals., 1950.	Per cent of total consumption.
Steam railroads	6	7	4.76
Manufacturing and mechanical	6	7	4.76
Street sprinklers	6	7	4.76
Lawn sprinklers	3	3½	2.38
Consumers not connected to sewers	9	10½	7.14
Leakage from mains and services	18	21	14.28
Totals	48	56	38.08

In the same way the average of the maximum rates of flow for the two moderately developed districts is found to be 26,025 gals. per acre daily, equivalent to a maximum rate of flow of 1,510,000 gals. per day from 57.9 acres. Domestic sewage at a maximum rate of 135 gals. per capita per day for a population of 2,810 corresponds to a maximum rate of 380,000 gals. per day, leaving 1,130,000 gals. per day as the maximum rate of flow from the purely commercial district. This corresponds to 19,500 gals. per acre daily.

No data exist to indicate the commercial sewage which may be contributed from the 133.3 acres of commercial area which is as yet largely undeveloped. It has been assumed that the maximum rate of flow of commercial sewage from this area may be taken as 12,000 gals. per acre per day.

Assuming that the relative proportions of highly developed, moderately developed and substantially undeveloped area will remain practically unchanged, the average maximum rate of sewage discharge from commercial areas, due solely to their use for commercial purposes, has been estimated as equivalent to 44,000 gals. per acre daily; or, in round figures, 40,000 gals. per acre daily, in addition to domestic sewage from the resident population and ground water.

From the estimated future commercial area of 721 acres the maximum rate of flow has been computed on the basis of 40,000 gals. per acre per day for commercial sewage; 750 gals. per acre per day for ground water, as hereinafter explained, and 135 gals. per capita per day for domestic sewage.

Quantity of Sewage from Industrial Areas.—As certain portions of the city are and will continue to be devoted to industrial enterprises from which large quantities of liquid wastes will be discharged into the sewers, an effort has been made to secure data which might aid in forming an estimate of the quantity for which provision should be made in the intercepting sewers.

It should be noted that there is always a tendency for certain classes of industrial plants, such as packing houses, dairies and the like, to discharge into the sewers large quantities of solid or heavy semi-fluid wastes. Precautions must be taken to prevent the entrance of such matter into the intercepting sewers, if necessary, by requiring all such plants to maintain settling tanks upon their branch sewers.

It is extremely difficult to determine the quantity of wastes which will be discharged from an area devoted to industrial purposes, as the amounts produced by manufacturing establishments vary enormously and because such an area is also developed for residential purposes to a considerable extent. Actual measurements were desired and were made for four districts which were selected for that purpose.

The Marshall Avenue District is probably more nearly a typical industrial area than any of the others, although the quantity of wastes per acre discharging into the sewers is probably greater than is to be expected from the industrial areas of the city as a whole. The conventional curve and other data for the district are shown in Fig. 1.

The effect of the industrial wastes in increasing the per capita rate of flow in this district is clearly shown by comparing the average rate of 363 gals. per day with that obtained in the Mitchell Avenue and Ross Run sewers in the residential district, where the average quantity of sewage was found to be 80 and 92 gals. per capita per day, respectively.

Quantities of Industrial Wastes.—In connection with the studies of the purification of sewage, a careful census of the industrial establishments of the city and of those lying north of the city in Mill Creek Valley has been taken. The quantity of water used in the several industries obtained from the canal, wells and other sources, and the quantity obtained from the municipal water supply, together with the estimated quantities of liquid wastes, were tabulated.

Nearly all the industrial plants are located in Mill Creek Valley and the McLean Avenue Sewer districts. The industrial water consumption in this region amounts to about 17,000,000 gals. per day. Not all the water so used is discharged in the form of wastes. The total amount of these wastes is estimated at 16,000,000 gals. per day, which is equivalent to 5,500 gals. per acre daily, the area under discussion being approximately 2,900 acres.

In view of the fact that nearly all of the plants run but 10 hours per day, the maximum rate of discharge of industrial wastes has been estimated on the assumption that 75 per cent of these wastes are discharged at a uniform rate during these 10 hours. The corresponding maximum rate of flow is 9,900 gals. per acre daily.

Similar estimates of the quantities of industrial wastes from smaller manufacturing districts tributary to the Ohio River show quantities equivalent to 6,000, 1,100 and 2,350 gals. per acre per day, the corresponding maximum rates of discharge being 10,800, 2,000 and 4,200 gals. per acre per day, respectively.

Giving due weight to the extent and the character of the development of the several districts, it is believed that a fair estimate of the maximum rate of discharge of industrial wastes into the intercepters is 9,000 gals. per acre per day. This quantity is assumed to be in addition to the domestic sewage from the resident population and the ground water.

Ground Water.—The quantity of ground water which reaches the sewers depends on the rainfall, the character of the soil and upon whether the sewers and their connecting drains are well or poorly built. Many of the older sewers were laid without precaution to prevent the infiltration of ground water, which even under the best conditions cannot be entirely avoided. The quantity of ground water thus entering the sewers, as previously explained, varies greatly from time to time and is usually highest after rains in the spring of the year.

It has been assumed, as previously explained, that a great increase in the area of the city will take place during the next 40 years. The estimated density of population over this large area is relatively small, being but eight persons per acre, whereas the density of population in a number of other large cities of this country is found to be from 15 to 25 persons per acre. The small number of persons per acre makes it evident that the number of miles of sewers per square mile of area is likely to be relatively small.

The report of the United States Census Bureau upon Statistics of Cities for 1907 contains data from which the number of miles of sewers per thousand inhabitants in cities having populations of more than 30,000 can be derived. From this information it appears that a well-sewered city of 720,000 persons should have about 900 miles of sewers, and it has therefore been assumed, in estimating the quantity of ground water to be provided for, that there will ultimately be that length of sewers contributing to the intercepters. Numerous measurements and estimates of the infiltration of ground water into sewerage systers indicate that 75,000 gals. per mile of sewer is a reasonable daily maximum. It has therefore been assumed that the total quantity of ground water which will eventually reach the intercepting sewers will be 67,500,000 gals. per day, from an area of 140 square miles, equivalent to 750 gals. per acre per day. This unit has been used in computing the quantity of sewage likely to reach the intercepters from each of the several sewer districts.

Storm Water.—It will be noted that in the foregoing discussion of quantity of sewage there is no allowance for storm water. In other words, when the city shall have attained the degree of development herein estimated as probable in 1950, if the quantities of domestic, commercial and industrial sewage and ground water correspond to the estimates, the intercepting sewers designed to take those volumes of sewage will flow full at the time of maximum sewage flow, and there will be no surplus capacity for storm water at that time.

Even then the flow of sewage would not fill the intercepters except during hours of maximum flow, so that during perhaps half of the day there would be a small surplus capacity. The first, and therefore the foulest, run-off from storms occurring at such times could therefore be taken into the intercepters.

The maximum rate of sewage flow estimated for the present time is but little more than half that to be expected in 1950, and after adding the ground water there would still be available at the present time about one-third the capacity of the intercepter at times of maximum average flow. That is to say, storm water could be received up to a quantity approximately equivalent to one-half the maximum rate of flow of sewage and ground water. Thus the first storm water flowing off from the streets and roofs would be conveyed with the sewage to the point of ultimate disposal.

As the years go on the surplus capacity available for storm water would grow less, until ultimately the maximum sewage and ground water flow would fill the intercepters, as stated above.

At the points where the combined sewers discharge into the intercepters adequate storm water overflows must be provided to convey to the nearest watercourse the excess storm water beyond the quantities which the intercepting sewers may be able to receive. The subject of storm water overflows and sewage flow regulators will be discussed in a subsequent article.

SPECIFIC DATA ON ADOPTED DESIGNS.

Sections.—Two forms of cross section will be employed on the new intercepters, namely, circular and semi-elliptical. The circular type will be used for sections 5 ft. in diameter and less. On all larger sections the semi-elliptical section will be used. In the larger sewers this latter section has the advantage of economy, since the quantity of concrete and excavation are both somewhat less than for a circular section of the same capacity. This type of section is more satisfactory structurally, also, since the flatter invert is more easily built than a large circular invert, and the reinforcing steel is more easily placed. Typical sections of both types are shown in Fig. 5 and the relations of the several hydraulic elements involved in the calculation of flow in the semi-elliptical section are shown in Fig. 6.

Profiles.—The elevations of the several trunk and branch sewers which must discharge into the intercepter govern its elevation and slope, at least within somewhat narrow limits. The design for the Mill Creek Intercepter has been made with the requirement that there shall be a scouring velocity of not less than 3½ ft. per second when the sewer is flowing full, and when the discharge is not retarded by high water at the outlet.

Conditions are complicated in the case of Mill Creek Intercepter by the frequent high-water stages of the Ohio River. Whenever the river reaches or passes the 20-ft. stage on the Broadway gage the effect of back water will begin to be felt in the intercepter, and when the river reaches the 30-ft. stage, if not before, the lower portion of the intercepter will be entirely submerged, and much, if not all, of the sewage will then be discharged through the storm overflows into Mill Creek.

When the velocity of flow is checked by the rise of the river deposition of silt will occur in the intercepter. It is desirable that when the river falls again as high velocities as are reasonably possible be obtained to scour out the accumulated deposits.

The average number of days per year when the Ohio River has been at or above any given elevation indicates that there have been on an average 122 days per year when the flow in the Mill Creek Intercepter would be checked, and 53 days per year when it would be almost entirely suppressed in the lower section, these periods corresponding to the 20- and 30-ft. stages of the river, respectively.

The Mill Creek Intercepter is 4 ft. in diameter in its smallest section. For this section the slope is 0.0015, and this slope continues until a size of 5 ft. 9 ins. is reached, where it changes to 0.0010. At the 7 ft. 3 in. section the slope again changes to 0.0006 and continues at that rate to the outlet where the size of the section is 8 ft.

The Duck Creek Intercepter is below the bed of the creek throughout its course and is therefore low enough to receive the tributary trunk sewers without difficulty. The outlet is well above the level of extreme high water in the river and consequently the intercepter will not be affected by fluctuations of river stage. This sewer ranges in diameter from 2 to 4 ft. and in slope from 0.0052 to 0.0012.

When built the Ohio River Intercepter will be substantially parallel to the Ohio River to intercept the present and future trunk sewers which would otherwise discharge directly into the river. No direct comparison of the several projects for the Ohio River Intercepter is possible, since none of the projects is complete without the inclusion of the corresponding scheme for sewage treatment and a definite plan for sewage treatment will not be adopted for several years to come.

Unit Prices.—In all estimates of cost, except those made in connection with the studies of sewage treatment plants, the following unit prices have been used:

Excavation (including refill, sheeting, pumping) for trench with bottom width of 8 ft. and less, per cu. yd.	$ 1.25
Excavation (including refill, sheeting, pumping) for trench with bottom width more than 8 ft., per cu. yd.	1.00
Plain concrete masonry, per cu. yd.	10.00
Reinforced concrete masonry in sections from 5 ft. 3 ins. to 8 ft. 3 ins. diameter, inclusive, per cu. yd.	15.00
Reinforced concrete masonry in sections greater than 8 ft. 3 ins. diameter, per cu. yd.	14.00
Duck Creek outfall sewer, 45-in. section in tunnel, complete, per lin. ft.	27.00
Mill Creek outfall sewer, 8-ft. section in tunnel, complete, per lin. ft.	55.00
Manholes, per lin. ft.	6.00

During the design of these sewers Mr. Victor T. Price was Director of Public Service and Mr. Henry M. Waite was Chief Engineer. Mr. H. S. Morse was Engineer in charge of all sewerage studies and Mr. Harrison P. Eddy was Consulting Engineer. Messrs. George W. Fuller and Allen Hazen collaborated with Mr. Eddy in reviewing and approving the matters considered in this article.

Roads and Streets

Cost of Operating Motor Trucks in Road Maintenance With Methods of Upkeep.

There is no equipment better adapted for the maintenance of bituminous surfaces and bituminous pavements, or for making roads, than motor trucks; and probably none is more generally in use for this purpose. A discussion by A. F. Masury in Vol. XL, pp. 1526-32, in the Proceedings of the American Society of Civil Engineers, gives facts, based on the actual operation and maintenance of trucks, which will enable the engineer and contractor to operate them to the best advantage in road work.

Arrangement of Space for Most Economical Storage.—In storing motor trucks, it has been worked out in practice that they can be handled best in a building of usual size by arranging them in two rows with an open aisle between; that it, the trucks are backed into place, with their fronts facing the aisle. This permits them to be taken out without any delay, and also allows them to be run into the aisle, where washing and drainage facilities should be provided.

In most buildings, posts are necessary, and should be toward the front of the trucks, but far enough back to allow the trucks to turn out into the aisle. The depth from the wall to the post should be at least 16 ft. 6 ins.; thus, the front wheels of trucks of most sizes will project beyond the posts and allow an easy and quick turning into the aisle. The aisle should be about 27 ft. wide in order to provide a practical turning radius for getting out or backing in. The posts should be about 22 ft. apart, so as to allow the storage of three large trucks between them. Where a large number are stored the proportioning of small to large trucks allows this arrangement to be worked out with the economical use of all space.

The size of the doors should be looked into very carefully, as trucks, especially of the van type, are now very large. In order to take those of the largest type, such as vans and those used for crackers and paper boxes, the doors should have a clearance of at least 12 ft. 3 ins., and should be double, opening outward. The clearance between the butts of the doors should be at least 10 ft. 9 ins. This allows sufficient space for the fastenings of the doors, without interfering with the trucks as they move in and out. There should be lights on the butts of the doors on each side, quite high up.

The elevators must also be considered carefully. They must be at least 24 ft. long and 11 ft. wide, in order to take the modern motor truck, and should have a carrying capacity of at least 7 tons, in order to accommodate the trucks when light. A heavier elevator, to take care of loaded trucks, is impractical, from a commercial standpoint; loaded trucks, are not usually garaged in such buildings, but might be accommodated on the ground floor. The elevator should have four guiding rails, and a speed of about 60 ft. per min.

TABLE I.—MOST IMPORTANT AVERAGE DIMENSIONS OF TRUCKS.

Type.	Capacity. Tons.	Wheel base. Ft.	Wheel base. Ins.	Extreme length. Ft.	Extreme length. Ins.	Length, back of cab. Ft.	Ins.		Ft.	Ins.	Extreme width, in ins.	Height of loading platform, in inches. Empty.	Height of loading platform, in inches. Loaded.	Clearance, in ins.
Type 1	1	12	0	16	0	9	2				68	36	32	9½
	2	13	0	17	0	9	2				68	38	34	9½
	3	13	6	20	6	11	0	to	11	6	87	42	38	10
	4	14	6	21	6	12	0	to	12	6	87	42	38	10
	5	15	6	22	6	13	0	to	13	6	87	44	38	10
	7½	16	6	23	6	14	0	to	14	6	87	44	38	10
Type 2	3	11	6	18	3	12	6				87	42	38	10
	4	12	6	30	3	14	6				87	42	38	10
	5	13	6	22	3	16	6				87	44	38	10
	7½	14	0	23	3	17	6				87	44	38	10
Type 3	5	11	10½	18	1	14	0				93½	46	40	7½
	7	13	4½	20	1	16	0				93½	46	40	7½
	10	14	2½	22	1	18	0				93½	46	40	7½
Power-dump truck, Type 1	5	11	9	16	5	8	3				87	44	39	10½
	7½	12	10	17	5	9	3				87	44	39	10½

Service.—There must be adequate facilities for filling the tanks with gasoline, and oil, and for the storage of the large quantities of lubricants used in the operation of such trucks.

To comply with fire prevention regulations, the gasoline must be stored underground, and must be piped to at least two outlets on each garage floor. There are two practical ways of raising it to each floor; by hydraulic power and by pressure. Either of these systems can also be used for cylinder oil and kerosene. There should be at least one outlet for kerosene and one for oil on each floor. Meters should be attached to all outlets, and provision should be made for locking them. At least one portable tank should be provided for each floor, as trucks must be filled during the hours when the garage is most crowded, and when, at best, only two can be filled from each outlet.

Oil and kerosene may be carried to the trucks from the outlets in measures, as comparatively small quantities are used.

TABLE II.—FACTORS IN THE COST OF OPERATION OF MOTOR TRUCKS.

Capacity. A	Average cost of chassis. B	Miles per hour. D	Driver's salary, per year. F	Garage, per month. G	Miles per day. H	Maintenance, per year. I	Gasoline, per year. J	Oil and grease, per year. K	Tires, per year. L	Interest, per life. M	Insurance, per life. N	Depreciation during life. O	Cost, per year. P	Miles of life at each mileage per year. Q	Life of truck, in years. R	Cost, per life. S	Cost, per day T	Tire cost, per mile. U	Gasoline, cost per mile. V	Miles of use, per year. W
1- ton	$1,963 (12)	19	$780	20	40	$120	$300	$120	$360	$408.75	$260.70	$1,626	$2,197.78	100,000	8.3	$18,241.74	$7.33	$0.03	$0.025	12,000
			830		60	240	450	190	540	231.75	147.00	833	3,106.78	85,000	4.7	14,601.86	10.36	0.03	0.025	18,000
			900		90	360	675	270	810	117.76	75.00	426	3,531.78	65,000	2.4	8,476.27	11.77	0.03	0.025	27,000
1½-ton	2,225 (12)	18	780	21	35	135	276	102	341	533.97	331.00	2,140	2,199.73	100,000	9.6	21,117.41	7.33	0.0325	0.0262	10,500
			830		50	270	395	150	488	317.06	202.00	1,270	2,698.73	85,000	5.7	15,922.51	9.00	0.0325	0.0263	15,000
			900		90	405	710	270	878	133.50	85.00	535	3,723.73	65,000	2.4	8,948.95	12.43	0.0325	0.0262	27,000
2- ton	2,918 (8)	17	780	22	35	150	288	75	393	663.33	461.00	2,660	2,361.44	95,000	9.1	21,489.10	7.87	0.0375	0.0274	10,500
			830		50	300	415	150	563	408.52	283.00	1,818	3,067.44	85,000	5.6	17,177.66	10.22	0.0375	0.0276	15,000
			900		80	450	660	225	900	246.96	136.00	789	4,026.44	65,000	2.7	10,877.39	13.42	0.0375	0.0275	24,000
3- ton	3,575 (6)	15	780	23	30	180	292	75	404	893.75	572.00	3,575	2,233.39	90,000	10.0	22,333.90	7.44	0.045	0.0324	9,000
			820		50	360	487	150	678	473.68	303.00	1,980	3,612.39	80,000	5.3	19,145.67	12.04	0.045	0.0324	15,000
			900		80	540	781	225	1,081	241.31	154.00	1,018	4,329.39	65,000	2.7	11,716.35	14.43	0.045	0.0324	24,000
4- ton	4,066 (3)	14	780	24	30	240	360	75	450	1,016.50	650.00	4,066	2,761.31	90,000	10.0	27,613.10	9.20	0.05	0.04	9,000
			830		50	480	600	156	750	538.94	344.00	2,140	3,666.31	80,000	5.3	19,431.44	12.22	0.05	0.04	15,000
			900		70	720	840	225	1,025	364.95	195.00	1,210	4,566.31	65,000	3.0	13,698.93	15.22	0.05	0.04	21,000
5- ton	4,500 (10)	12	800	25	30	240	406	90	540	1,125.00	720.00	4,500	2,920.50	90,000	10.0	29,205.00	9.74	0.06	0.045	9,000
			900		50	480	678	180	900	596.25	311.00	2,380	4,072.50	80,000	5.3	21,184.25	13.58	0.06	0.045	15,000
			1,000		70	720	948	262	1,260	337.50	216.00	1,350	5,124.50	65,000	3.0	15,373.50	17.08	0.06	0.045	21,000
7½-ton	5,400 (2)	9	800	26	25	255	395	94	750	1,525.50	976.00	5,400	3,367.40	85,000	11.3	38,051.62	11.22	0.10	0.0526	7,500
			900		40	510	630	150	1,200	837.60	535.00	2,348	4,463.40	75,000	6.2	27,673.08	14.88	0.10	0.0525	12,000
10- ton	5,500 (1)	7	800	28	25	270	468	94	1,200	1,553.75	994.00	5,500	3,943.50	85,000	11.3	44,561.55	13.15	0.16	0.0624	7,500
			900		40	540	750	150	1,920	852.50	545.00	3,410	5,371.50	75,000	6.2	33,303.30	17.90	0.16	0.0625	12,000

Column letter in Table II.		Formulas.
A	Capacity.	
B	Average cost of chassis, based on list prices of the number of manufacturers indicated by the figures in parentheses.	
D	Miles per hour.	
F	Driver's salary, per year.	
G	Cost of garaging, per month.	
H	Miles per day.	
I	Maintenance (small repairs, etc.) per year.	
J	Gasoline, per year.	
K	Oil and grease, per year.	
L	Tires, per year.	
M	Interest, per life.	$M = R\frac{B}{2} \times 5$ per cent.
N	Insurance, per life.	$N = R \times (80\%$ of $B) \times 2$ per cent.
O	Depreciation, per life.	$O = \frac{B}{10\% \text{ of } R}$
P	Cost, per year.	$1 + F + 12G + 1 + J + K + L + \frac{M}{R} + \frac{N}{R} + \left(O = \frac{B}{10}\right)$
Q	Miles of life at each mileage per day truck is assumed to run.	
R	Years of life.	$R = \frac{Q}{W}$
S	Cost, per life.	$S = P \times R$
T	Cost, per day.	[illegible]
U	Cost of tires, per mile.	[illegible]
V	Cost of gasoline, per mile.	$V = \frac{J}{W}$
W	Miles per year.	$W = H \times 300.$

Transmission oil, cup grease, wood alcohol (for use in the radiators in cold weather), and other material and accessories may be stored in a room on the floor with the trucks.

There should be an efficient system for keeping account of the exact quantities of supplies consumed by each machine.

A dirty truck (and trucks collect dirt with astonishing rapidity) is much harder to keep in good order than a clean one, as dirt not only gets between moving parts and into bearings and causes wear, but it prevents the discovery of minor troubles until they have become major ones, and makes repairs a difficult and distasteful job if they are discovered. Therefore, trucks should be washed at night. For this purpose a regular force should be employed, its size depending on the number of cars to be washed. Four washers and two polishers can take care of fifty cars.

Drivers are not always to be depended on to make repairs; their hours are long, and they have little time for them during the day's run. For this reason, if six or more trucks are being operated, it will be well to have a night repair man on duty, with one or two helpers. As the trucks come in, each driver should report any trouble with his machine. The repair man can also examine the trucks to discover loose bolts, and can grease them up.

Truck Sizes.—Trucks may be divided into three classes, according to the arrangement

TABLE III.—TURNING CIRCLES OF MOTOR TRUCKS.

Type.	Capacity.	Wheel base. Ft.	ins.	Diameter of turning circle, in ft.
Type 1	1- ton	10	6	47
		11	6	53
		12	6	55
Type 1	1½-ton	10	6	47
		11	6	53
		12	6	55
		13	6	60
Type 1	2- ton	10	6	47
		11	6	53
		12	6	55
		13	6	60
Type 1	3- ton	12	6	55
		13	6	59
		14	6	61
Type 1	4- ton	12	6	55
		13	6	59
		14	5	61
Type 1	5- ton	12	6	57
		13	6	62
		14	6	68
Type 1	7½-ton	12	6	57
		13	6	62
		14	6	68
		15	6	71
Type 2	3- ton	11	0	47
		11	6	49
		12	0	51
Type 2	5- ton	11	6	53
Tractor	3- ton	10	4	40
Tractor	5- ton	10	8	41

The trailer will usually cut under.

TABLE IV.—TIRE EQUIPMENT FOR TRUCKS

(1) Truck capacity.		(2) Axle load.	(3) Size of tires		(4) L / W	(5) Speed, in miles per hour.	(6) Mileage.	(7) Consumer's price
1-ton	Front	[illegible]	36-4	single	18	16.5	7,000	$54.15
	Rear	5,200	36-5	single	28.8			70.45
	Rear	[illegible]	36-2½	dual	28.8			51.30
1½-ton	Front	2,800	36-4	single	19.4	15	7,000	54.15
	Rear	6,300	[illegible]	dual	[illegible]			68.80
2-ton	Front	3,060	36-4	single	[illegible]	14	7,000	54.15
	Rear	[illegible]	[illegible]	dual	28.6			83.67
3 ton, Type 1	Front	4,300	36-5	single	23.9	12½	7,000	70.45
	Rear	10,800	[illegible]	dual	[illegible]			108.25
	Rear	10,800	42-5	dual	[illegible]			126.17
3-ton, Type 2	Front	[illegible]	[illegible]	single	[illegible]	12½	1,000	70.45
	Rear	[illegible]	[illegible]	dual	[illegible]			108.35
	Rear	[illegible]	[illegible]	dual	[illegible]			126.27
4 ton	Front	[illegible]	[illegible]	single	22.9	11	7,000	87.97
	Rear	[illegible]	[illegible]	dual	36.0			138.90
	Rear	12,9[illegible]	[illegible]	dual	[illegible]			172.40
5 ton, Type 1	Front	[illegible]	[illegible]	single	29.4	10	7,000	70.45
	Rear	15,200	[illegible]	dual	42.2			138.90
	Rear	[illegible]	[illegible]	dual	36.2			178.40
5-ton, Type 2	Front	[illegible]	[illegible]	single	34.4	10	7,000	87.97
	Rear	[illegible]	[illegible]	dual	[illegible]			138.90
	Rear	[illegible]	[illegible]	dual	36.7			172.40
7½ ton	Front	6,175	36-6	single	28.6	7½	7,000	87.47
	Rear	[illegible]	36-6	dual	[illegible]			175.97
	Rear	[illegible]	[illegible]	dual	[illegible]			205.95

of the motor and the driver's seat:

Type 1.—This is the usual touring car type; the motor is over the front axle, the driver and control are behind the motor.

Type 2.—In this type the motor is over the axle; the driver and control are directly above the motor.

Type 3.—The motor is over the front axle; the driver and control are on one side of the motor, and the helper is on the other side.

Types 2 and 3 conserve the length of the chassis, as less space is taken for the driver and control, thus allowing a longer loading platform in proportion to the wheel base than Type 1.

The most important average dimensions for each size of truck are given in Table I; and the diameters of turning circles are given in Table III.

Factors in Cost of Operation of Motor Trucks.—Table II shows the cost of operation of trucks of standard capacities running at different mileages per day, and the factors in detail of each item of upkeep. These figures are taken from actual costs of operating trucks, and are nominal in every way. Interest is reckoned at 5 per cent on half the cost of the truck; insurance at 2 per cent on 80 per cent of the cost; and amortization at 10 per cent.

In Table II the cost of the body is left out intentionally, as it usually represents the amount of discount given by the manufacturer, and it needs but few repairs.

Tire Data.—The data on tire equipment in Table IV show the average of the requirements and guaranties of the different tire manufacturers, and also the average cost of tires to the consumer. Column 2, the axle load, gives the average weight on the front and rear axle of trucks of each size. Column 3, the size of tire, shows the sizes usually recommended by tire manufacturers for the respective loads carried on each axle. Column 4, $\frac{L}{W \times C}$, means the load on the axle, in pounds, divided by the width of the tire (the width being taken at the place where the tire meets the ground); C is the circumference of the wheel. The figures in this column are pounds.

This factor takes into account the size of the wheel, and therefore reconciles the fact that a larger wheel can have a tire of a smaller size, and is a better factor to use in comparing tire sizes than the weight, in pounds per square inch, of tire in contact with road.

Column 5 shows the normal speed of trucks of each capacity. Column 6 shows the guaranteed mileage by the tire manufacturers. Column 7 shows the average price at which tires can be bought by the consumer.

Costs of Roads Constructed Under the Supervision of the Michigan Highway Commission.

Contributed by K. I. Sawyer, Lansing, Mich.

(Note.—Further references to gravel and macadam road construction in Michigan will be found in the issues of Engineering and Contracting for Dec. 28, 1910; Jan. 4, Feb. 21, March 8, 15, 1911; March 13, May 8, 22, 29, 1912.)

During the earlier history of Michigan, when railroad facilities were poor, quite a few so-called "state roads" were built as a stimulus to development. These were largely financed by land grants. The coming of the railroad with its cheaper methods of intercommunity intercourse did away with the road as an intercommunity necessity and the road became a purely community utility. As the country developed, better routes between communities became essential and 21 years ago a system of road procedure was provided for by which the intercommunity needs could be handled by county officials with the county as a unit, and the community interests were vested in the township authorities. Still further development made better roads and better construction methods necessary, and in 1905 the Michigan legislature enacted a statute creating a State Highway Department and providing means for rewarding the construction of good roads. This law recognized the principles mentioned and considered the road, as it then was, a community utility.

TABLE I.—REWARDS PAID PER MILE FOR DIFFERENT CLASSES AND WIDTHS OF ROAD (WIDTH APPLIES TO SURFACING).

	9 ft.	10 ft.	11 ft.	12 ft.	13 ft.	14 ft.	15 ft.	16 ft.	Required depth of metal.
*Class A	250	275	300	325	350	375	400	425	5 ins to 8 ins.
Gravel	500	550	600	650	700	750	800	850	8 ins.
†Class C	750	825	900	975	1,050	1,125	1,200	1,275	7 ins.
‡Class D	750	825	900	975	1,050	1,125	1,200	1,275	7 ins.
Macadam	1,000	1,100	1,200	1,300	1,400	1,500	1,600	1,700	6 ins.
Concrete	1,000	1,100	1,200	1,300	1,400	1,500	1,600	1,700	Specifications to be approved by State Highway Commissioner.
Brick	1,000	1,100	1,200	1,300	1,400	1,500	1,600	1,700	

*Several types (i. e., sand, clay, etc.) for light traffic.
†Gravel top course, stone or slag bottom course.
‡Slag or stone top course, slag or gravel bottom course.

TABLE II.—ROADS BUILT UNDER STATE SUPERVISION 1905-1914.

Kind.	Width for which rewarded.	Miles.	State reward.	Total cost.
Class A	9 ft.	52.131	$ 13,033.00	$ 88,643.27
Gravel	9 ft.	1,474.527	737,264.00	3,215,777.51
Macadam	9 ft.	574.521	524,521.00	2,559,495.93
Concrete	9-16 ft.	64.886	97,558.00	947,573.13
All other types	....	189.256	300,032.00	1,010,280.09
Totals	....	2,355.321	$1,672,408.00	$7,821,769.93

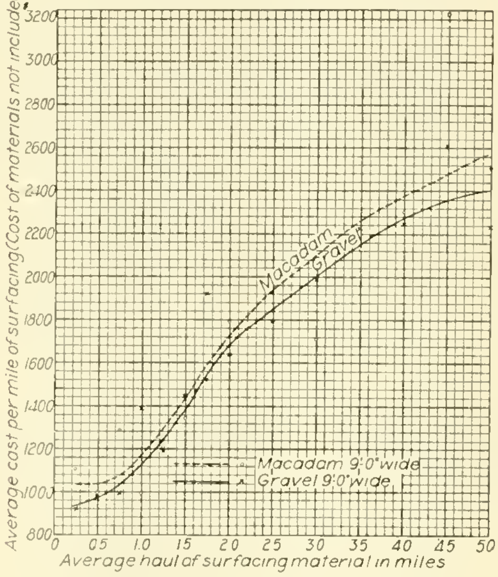


Fig. 1. Curve Showing the Effect of Length of Haul on Cost of Surfacing in Michigan.

Method of Rewarding.—It provided a flat rate reward, or contribution toward the cost after the road was completed, for the first five listed kinds of road shown in Table I, in amount equal to that shown for the 9-ft. width. These rewards were payable either to townships or counties for each mile of road constructed in accordance with the requirements of a minimum specification. The function of the Highway Department was primarily the educating of road officials in construction methods, the supervising of the rewarded roads and the collection and transmittal of data and information relative to roads.

Very soon after this development the constantly increasing number of automobiles and trucks with their greater radius of economic highway transportation placed a new aspect on the road as a transportation factor.

Trunk Lines.—While there is little doubt that in practically all cases the chief benefit of any road is to the community through which it passes, it is now also apparent that certain roads are of considerable general benefit as through routes for traffic. The legislature of the state took cognizance of these facts and a year ago named certain through or trunk roads as comprising a state trunk line system, specifying that for each mile of trunk road built a double reward should be paid.

It further provided that the plans and specifications for roads on these routes be made by the State Highway Department, thereby insuring uniformity of construction. It also provided that the state build the bridges on

TABLE III.—DISTRIBUTION OF COST OF GRAVEL ROADS ACCORDING TO LENGTH OF HAUL (WIDTH 9 FEET).

	(1)	(2)	(3)	(4)	(5)	(6)	
Average haul.	Length. Miles of road.	Cost. Ditches and grade.	Cost. Surfacing	Cost. Gravel.	Cost. Bridges and culverts.	Yards of gravel used (loose measure).	Total cost.
0.250 miles.	1.77	$ 1,613.21	$ 1,635.21	$ 296.00	$ 599.60	2,370	$ 4,144.02
0.500 miles.	3.36	990.00	3,302.55	499.27	267.00	5,050	5,058.62
0.750 miles.	5.79	2,044.02	5,817.14	1,033.25	972.00	9,758	9,866.41
1.000 miles.	5.25	3,485.00	7,340.22	4,648.33	503.00	8,865	15,976.55
1.250 miles.	3.00	1,161.36	3,567.11	498.00	275.05	5,020	5,501.52
1.500 miles.	5.17	1,896.00	7,492.26	807.45	291.80	8,125	10,490.51
1.750 miles.	4.39	2,177.08	8,465.73	680.60	134.56	7,539	11,457.97
2.000 miles.	5.59	2,656.07	9,070.53	919.85	764.88	9,269	13,411.33
2.500 miles.	5.98	3,246.32	10,054.71	1,033.85	260.56	10,580	14,601.44
3.000 miles.	11.54	4,093.22	21,449.93	2,606.30	616.49	17,895	28,765.94
3.500 miles.	4.75	2,199.18	10,601.53	973.50	88 50	8,231	13,862.71
4.000 miles.	1.97	1,861.25	11,191.95	1,481.00	....	8,411	14,534.20
4.500 miles.	2.08	1,005.81	3,754.49	651.38	31.33	3,418	5,443.01
5.000 miles.	4.78	2,050.90	10,644.19	795.52	111.50	8,342	13,602.11
	68.12	$30,479.42	$114,387.35	$16,924.30	$4,925.27	112,873	$166,716.34
Average per mile...		$455.46		$247.34		1,649	$2,436.58

TABLE IV.—DISTRIBUTION OF COST OF MACADAM ROADS ACCORDING TO LENGTH OF HAUL (WIDTH 9 FEET).

	(1)	(2)	(3)	(4)	(5)	(6)	
Average haul.	Length. Miles of road.	Cost. Ditches and grade.	Cost. Surfacing.	Cost. Stone.	Cost. Bridges and culverts.	Yards of stone used.	Total cost.
0.250 miles.	3.01	$ 1,439.71	$ 3,372.30	$ 6,072.49	$ 473.09	4,955	$ 11,357.89
0.500 miles.	2.55	968.62	2,539.00	4,063.25	43.90	3,524	7,614.77
0.750 miles.	4.32	1,735.40	5,671.33	6,719.74	143.60	7,023	14,270.07
1 000 miles.	3.73	1,237.00	3,659.60	7,318.21	136.47	5,647	12,351.28
1.250 miles.	4.12	2,263.01	4,932.82	9,462.71	434.20	7,283	17,092.74
1.500 miles.	4.06	1,665.03	5,526.10	10,115.93	240.34	8,127	17,547.40
1.750 miles.	5.41	3,158.55	8,138.70	13,196.16	623.35	9,212	25,116.76
2.000 miles.	4.48	2,075.60	8,017.33	12,924.72	282.42	7,790	23,300.07
2.500 miles.	4.72	2,115.39	8,410.08	12,310.87	45.00	7,955	22,881.34
3.000 miles.	3.58	1,841.52	6,211.33	7,194.03	350.75	5,670	15,597.63
3.500 miles.	3.58	805.86	5,552.09	6,018.64	32.58	4,180	12,409.08
4.000 miles.	4 23	1,842.62	9,736.10	12,398.77	138.43	8,103	24,115.92
4.500 miles.	2.50	842.49	8,087.69	5,908.00	640.92	3,985	15,479.10
5.000 miles.	2.04	817.27	5,020.03	4,817.70	172.34	3,032	10,827.31
	52.33	$22,808.07	$84,808.07	$118,521.22	$3,757.39	86,486	$229,961.39
Average per mile...		$435.85		$2,264.88		1,653	$4,394.44

these through or trunk routes and provided for the organization of a bridge department.

Provision for Maintenance.—One further provision is worthy of note in that it offsets one of the chief drawbacks of reward methods of road construction, that is the provision of an annual repair reward for keeping rewarded roads in repair, with the option by a community of receiving no further benefits from the state if the rewarded roads are not kept in repair.

TABLE V. — SUMMARY OF COSTS OF 68 MILES OF GRAVEL ROAD.

	Average per mile.	Average per sq. yd. surfaced
Yards gravel	†1,619	†0.312
Grade shaping and drainage	$ 455.46	$0.086
*Gravel	247.31	0.046
Culverts, etc.	71.93	0.013
Surfacing (average of totals)	1,661.85	0.316
Totals	$2,136.58	$0.46
Total length (miles)		68.42
Total sq. yds. surfaced		361,278
Total cu. yds. gravel used		112,872

†Cu. yds. *Includes loading and hauling.

Upon the completion of each piece of road the department has required a cost report of the job before paying the reward. From these reports the figures herein given have been derived.

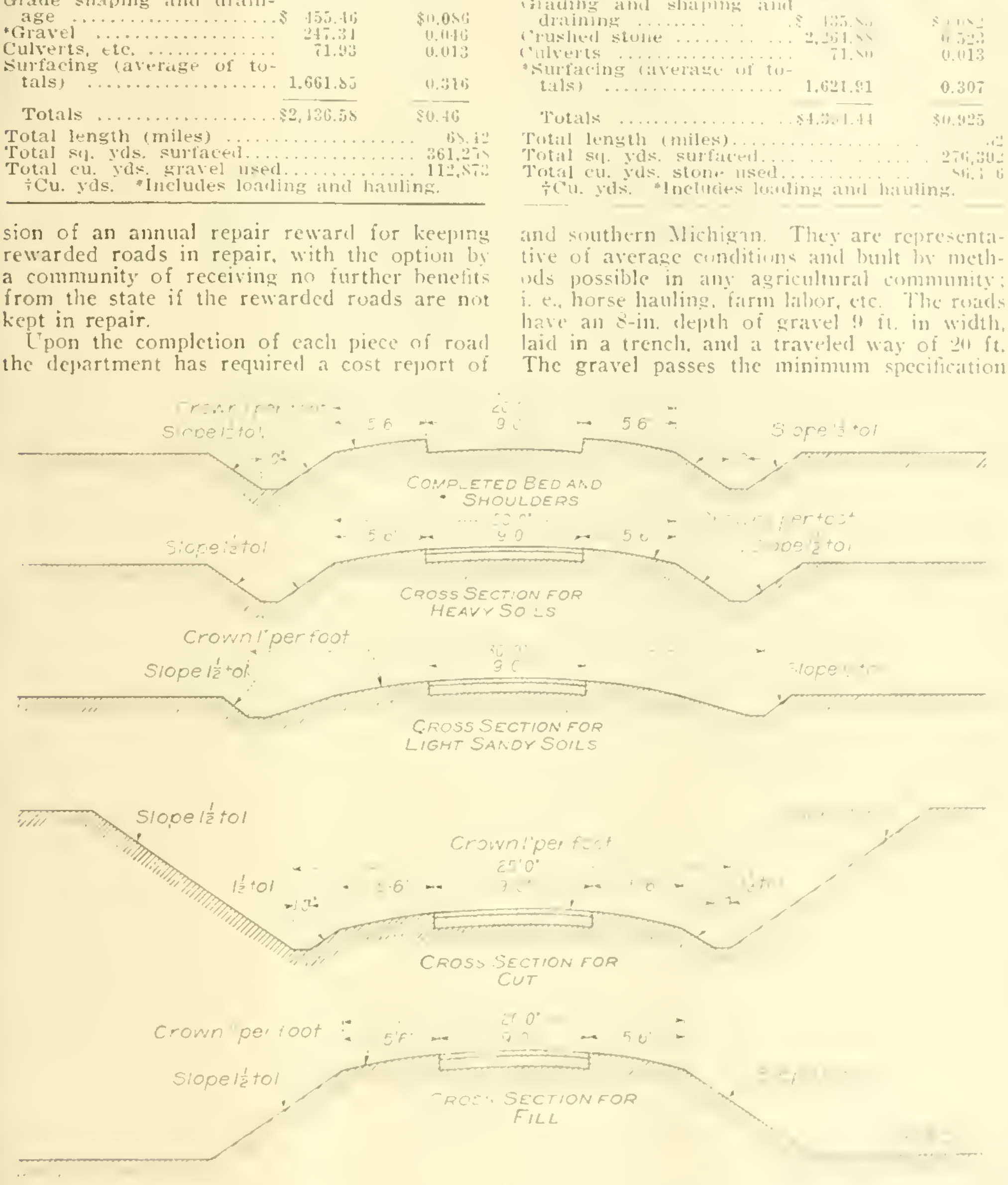


Fig. 2. Cross Sections Used for State Reward Roads in Michigan.

There is a great diversity in construction methods and plant used in their work, and with a wide range in conditions from suburban streets to roads through uncut forest and in the Porcupine Mountains, it is readily seen that state averages would mean but little in a cost discussion. For this reason, 59 jobs aggregating 68.42 miles of 9-ft. gravel roads, three or more of which have the same average haul, are added together under each classification of average haul in Table III.

TABLE VI.—COSTS PER SQ. YD., ARRANGED BY LENGTH OF HAUL OF GRAVEL.

Av. haul.	Per sq. yd. surfacing.	Per sq. yd. complete. (Surfacing plus $0.145.)
½ mile	$0.186	$0.331
1 mile	0.261	0.406
2 miles	0.305	0.450
3 miles	0.353	0.498
4 miles	0.428	0.573
5 miles	0.422	0.567

The totals as shown in Table II are of value primarily as showing the popular choice of types and the extent of construction. Since July 1, 1913, about 600 miles have been completed.

The roads used in compiling Table III lie in 12 of the agricultural counties of central and southern Michigan. They are representative of average conditions and built by methods possible in any agricultural community; i. e., horse hauling, farm labor, etc. The roads have an 8-in. depth of gravel 9 ft. in width, laid in a trench, and a traveled way of 20 ft. The gravel passes the minimum specification of containing 60 per cent by weight of stone retained on a ⅛-in. mesh screen and passing a 1½-in. ring.

TABLE VII.—SUMMARY OF COSTS OF 52 MILES OF MACADAM ROAD.

	Average per mile.	Average per sq. yd. surfaced.
Stone	†1,653	†0.313
Grading and shaping and draining	$ 435.86	$0.082
Crushed stone	2,261.88	0.523
Culverts	71.80	0.013
*Surfacing (average of totals)	1,621.91	0.307
Totals	$4,391.44	$0.925
Total length (miles)		52
Total sq. yds. surfaced		276,302
Total cu. yds. stone used		[illegible]

†Cu. yds. *Includes loading and hauling.

The cost of grading includes the preparation of the subgrade and providing drainage. In column 1 the cost of the gravel used either in the pit or on cars, including freight. In column 5 the cost of drainage structures. In column 6 the number of cubic yards of gravel used and in column 3 the cost of doing all surfacing work, including loading, hauling, spreading, rolling and superintending.

TABLE VIII.—COSTS PER SQ. YD. AVERAGED BY LENGTH OF HAUL OF STONE.

Av. haul.	Per sq. yd. surfacing.	Per sq. yd. complete. (Surfacing plus $0.618.)
½ mile	$0.189	$0.807
1 mile	0.187	0.805
2 miles	0.349	0.967
3 miles	0.328	0.946
4 miles	[illegible]	[illegible]
5 miles	0.466	1.084

A like classification, Table IV, is shown for macadam road having 6-in. stone, 9 ft. in width on a 20-ft. roadway. For this table 53 jobs were used, aggregating 52.33 miles of road located in six counties in the same district as the gravel roads in Table III.

Haul.—All costs which are affected by the haul are included in column 3 of each table. To better comprehend the affect of haul the graphical analysis shown in Fig. 1 was made plotting the costs per mile from column 3 as ordinates and the average haul in miles as abscissae. It is readily seen that the derived curves are generally parallel and that there is considerable uniformity in the location of the joints for gravel and considerable variation in the macadam, which leads to the suppositon that gravel costs can be predetermined with greater accuracy, and that the macadam costs have other leading considerations besides the length of haul, which is, of course, true.

One point worthy of note is that the differences do not equal the cost of rolling and sprinkling of macadam, which costs for macadam averaged $292 per mile on the jobs considered. This discrepancy probably arises from the fact that the gravel for most part was in the pit and required some work before loading, whereas the cost of the stone includes all work up to the time of loading.

Surfacing Material Used.—Column 6 offers an interesting figure in both Tables III and IV. It shows that on these jobs 1,649 cubic yards of gravel measured in the wagon were used per mile on a road built under specifications calling for 1,174 cu. yds. of packed gravel and that 1,653 cu. yds. of stone (2½-in., 1½-in. and binder), measured separately in wagons, were used for each mile of road built, the specifications requiring 880 cu. yds. of compacted stone. Both of these amounts are 10 per cent in excess of what it has been customary to consider as the requisite amount of material.

Tables V and VI give some of the unit costs for gravel roads and Tables VII and VIII give unit costs for macadam roads.

Methods and Cost of Constructing a Bituminous Gravel Concrete Surface.

The use of gravel aggregate in the construction of a bituminous concrete surface in New York state is discussed by S. J. Stewart in the Proceedings of the American Society of Civil Engineers, Vol. XL, p. 1088.

The writer designed a pavement consisting of a mixture of asphalt and gravel in the proportion of 1 cu. yd. of loose gravel to an average of 20 gals. of asphalt, the gravel containing not less than 10 per cent of clay. The gravel was bank run, the largest particles of which were 2 ins. in the longest dimension, containing sufficient fines to fill the voids partly. The bitumen was a fluxed natural asphalt with a penetration between 10 and 13 mm. when tested for 5 seconds at 77° F., on a No. 2 needle weighing 10 grammes.

The gravel was heated in a mechanical revolving dryer with a temperature of more than 225° F., after which the asphalt, heated to not less than 275° F., was added; then the mixture was placed in a revolving mixer until thoroughly and completely coated with bitumen. The mixture, at not less than 225° F., was spread on the prepared bottom course with shovels from dumping boards and raked to a uniform surface with hot rakes, after which it was rolled with a self-propelled roller weighing at least 10 tons, until it was thoroughly consolidated.

On some sections of this pavement there was [illegible] ½ in. of gravel screenings containing not less than 10 per cent of clay which was saturated with water and rolled thoroughly and continuously until a clay mortar had been obtained. This process filled all the surface interstices with a gritty and adhesive substance which made the road practically "non-skid." In a short time the traffic drove away all surplus screenings, leaving a mosaic surface.

A different method of treatment was applied to this form of pavement in the more thickly settled communities. After the mixture had been rolled, it was covered with a coat of hot oil as a seal coat, this seal coat being applied in November. In the following spring, under the action of the sun, the road bled to some extent, and became so sticky in a few places that the oil adhered to wagon wheels, which pulled up some of the top course. This condition could have been avoided if the authorities in charge had covered the pavement with sand or gravel screenings as soon as it became apparent that the hot oil had a tendency to bleed. No material harm was done, however, as the continuous traffic carried sufficient sand and dirt on the pavement so that in a short time the stickiness of the oil had disappeared.

Such treatment obviates the general complaint against the so-called stone-mixing method pavement where a greater quantity of asphalt is used and where the seal coat is of the same consistency as the asphalt binder in the top course proper. These objections arise from the hardness and slipperiness of the surface, which, during the greater portion of the year, make it undesirable for horse traffic because of the former characteristic and to motor traffic on account of the latter.

It is suggested that, where the hot oil is omitted at the time of the original construction, the mosaic surface be covered with a hot oil treatment of ½ gal. per square yard during the following year. With the hot oil as a squeegee course, in place of the seal coat of the same consistency of the bitumen, the pavement proper gives a less hard and slippery surface, which is desirable on country highways.

The writer's experience has convinced him that large plant, costing from $5,000 to $8,000, is not necessary in order to construct this pavement successfully, and that equally good results can be obtained from the use of small mixers costing from $1,500 to $2,000.

This pavement without the oil treatment cost about $0.85 per square yard, and with an oil seal coat about $0.90 per square yard for a pavement 2½ ins. in depth. This cost compares favorably with similar figures for mixing-method pavements in other parts of New York state.

The writer quotes from a statement attributed to a superintendent of highways, of New York state, relative to costs of similar pavements of graded stone material, as follows:

Item.	Cost per sq. yd.
1½ in., California asphalt	$1.20
2 in., Topeka	1.20
2 in., Warrenite	1.30
2 in., bitulithic	1.00

The gravel used in this pavement cost approximately $2.35 per cubic yard f. o. b. destination, but where material can be obtained near the site of the works the cost of $0.85 per square yard can be materially reduced.

From observations of mixing-method pavements laid in New York City and vicinity, with graded stone as a mineral aggregate, it is ventured that the percentage of disintegration is as great, if not greater, than in the bituminous gravel pavement laid during the same period of time. In fact, after 1 year's wear, out of a total of 112,000 sq. yds., or 0.01 of 1 per cent less than 45 sq. yds. of this gravel mixing-method pavement had disintegrated.

The road which received the treatment of hot oil in a squeegee course was dug up in many places during the year following its completion for investigating purposes, and, where removed, the material thrown back retained its vitality to such an extent that the top course healed itself so that a casual observer could not discover where the pavement had been disturbed.

Cost of Road Maintenance in Maryland.

Maryland has had a system of state road maintenance for several years, the work being divided into two classes: General maintenance by the patrol system, and oiling macadam roads. The cost of this work as given in the last report of H. G. Shirley, chief engineer of the Maryland Highway Commission, shows that the upkeep on comparatively new macadam roads is a considerable item of expense. It is possible, however, that the total cost of oiling should not be charged to maintenance since the quality of the road is materially bettered thereby. A portion of this expense should be charged to the cost of construction, or properly to betterment of surface.

Under present traffic conditions the maintenance of modern roads is not less important than their construction, especially or the latter can be deferred and the former cannot.

This fact has been consistently recognized by the commission, which has been alert in keeping the roads up to the highest standard, so that they compare today very favorably with those of other states. With the completion of the system in sight (725 miles of the 1,285-mile system have been completed), much care and study is being given to perfecting an organization capable of taking care of it in the future, properly and at a minimum cost. Maintenance may be roughly divided into; oiling, and patrol service, which subjects will be considered separately.

Oiling Roads.—A macadam road cannot be said to be finished until it is oiled, nor can it be preserved without oiling. It really should be oiled before being accepted by the state, but it seldom happens that the contractor is equipped to do this kind of work. In theory, macadam roads should be completed in the spring or early summer, so as to be oiled as soon as the weather is warm enough to secure proper penetration of the oil. In practice, they are generally completed in the late fall or winter, when it is too cold to properly apply the oil, which does not penetrate, but injures the road by picking up the surface. The public then insists upon the road being opened to traffic without this protection until the next summer, by which time it has frequently picked up and commenced to go to pieces. It then has to be rolled down and put in shape, at additional expense, before it can be oiled. This objection does not apply to concrete roads, and it will be obviated when construction is over.

In 1910, 38 miles were oiled, 140,000 gallons of oil used, and covered almost entirely with local sand and gravel at small cost. In 1911, 100 miles were oiled, 400,000 gals. of oil used, all being covered with local sand and gravel, except about 20 miles. In 1912, 182 miles were oiled, 625,000 gals. of oil used, and covered with 2,500 tons of purchased trap rock or limestone chips. This work had to be started late in the season, and the only equipment available consisted of several horse-drawn distributing wagons. While local sand or gravel could generally be had for covering at slight expense, it was found that they were generally of poor quality, and that much better results were secured by importing stone chips, although this added materially to the cost of the oiling.

In 1913, 325 miles were oiled, 950,000 gals. of oil used, and covered with 12,000 tons of purchased stone chips. The results this year were much more satisfactory, especially on the eastern shore and in central Maryland. This was brought about by letting contracts for the oiling in 16 counties to two large concerns in the business, and by purchasing an oiling motor truck for such work as could not be given out by contract. This made seven large pressure motor trucks operating in the state at one time. The purchased truck was made convertible, with two beds, one for oiling and the other for hauling stone, chips, gravel, etc., when not needed for oiling. Bought chips or washed gravel were used almost exclusively for covering, except in a few cases, where a satisfactory local gravel could be found.

In connection with this class of work a discussion of road maintenance by W. H. Fulweiler, in the Proceedings of the American Society of Civil Engineers, pp. 470-1, is of interest.

A contract involving 34 sections of road on the eastern shore of Maryland amounting to 118 miles (976,071 sq. yds.) of road was made. Under the terms of this contract the state furnished and applied the necessary stone covering and the contractor agreed to sweep untreated surfaces and surfaces already treated with bituminous materials at a unit price per square yard and to accept a unit price per gallon for bituminous material applied. Work was to be completed by July 31 under a penalty clause of $25 per day for exceeding this time limit and a further penalty of the same amount for each day's delay in operation caused by breakdown or failure of material to arrive. The state, in turn, was to pay the contractor $25 per day's delay in application of bitumen caused by the state's forces not covering it.

TABLE I.—SUMMARY OF MOTOR OILING OPERATION FOR 1912 AND 1913.

	1912.	1913.
Sundays and holidays	21	20
Repairs and delays	22	6
Rain	5	9
Transit	34	18
Applying material	57	72
Total elapsed days	138	125
Percentage of days worked	66%	72%
Percentage of days material applied	41.5%	57.5%
Average No. of gallons applied per day	2,860	4,760
Average No. of square yards treated per day	8,230	13,550

The time is given in truck-days.

Stone was purchased by the state and delivered in piles along the side of the road in the early spring when the price of teams was low. There were applied 342,785 gals. of material at a rate of ⅕ to ½ gal. per square yard, averaging 0.366 gals. per square yard. This was covered with stone chips averaging 12 lbs. per square yard.

Two motor trucks commenced work May 28. One truck finished half the work on July 18 and the other the remainder on July 30. The state maintained a gang of 12 men applying stone chips under an inspector with each truck, the contractor sweeping the roads and furnishing and applying the binder.

Practically this same form of contract was in force during 1912, but during that year the work was not laid out according to schedule, and the trucks frequently had to make long jumps between the different pieces of work. This caused considerable loss of time, which was furthermore augmented by difficulty in securing proper labor. As the result of the experience of 1912, the work was laid out more systematically in 1913, and men composing the covering gang in one case were held throughout the season. The effect of having experienced men for covering is well brought out in Table 1 which summarizes the operations of the two years.

In 1913 the decrease in time lost in transit, repairs, and delays, is quite striking, as is the average increase in the quantity applied per day.

In 1914, if satisfactory specifications can be prepared, the innovation will be made of dividing the state up into sections and calling for bids by advertisements during the winter for oiling by responsible companies. The oiling truck will be used for special work and for piecing out. It is anticipated that by this method oiling will be done much better, cheaper and more expeditiously than in preceding years.

TABLE II.—COST OF OILING MARYLAND ROADS BY YEARS.

Year.	Miles oiled.	Gals. of oil used per mile.	*Tons of chips used per mile.	Average cost per mile.
1910	38.40	370	43	$374.82
1911	100.39	388	102	363.84
1912	182.04	342	138	350.45
1913	326.61	295	364	346.59

*Note.—Local sand and gravel used in addition to stone chips.

Patrol Service.—A patrol system of maintenance has been adopted, and is being developed, extended and improved, but much remains to be done on this organization to perfect it. It requires experience, instruction and special knowledge to properly patch, repair and maintain a macadam road, and it will take several years to secure and train competent patrolmen to do this.

Each patrolman has been equipped with a distinctive cap to be worn in cool weather, a conspicuous number on his arm, and a red flag, which he is required to set up alongside the road during working hours, when on or off his job.

Several months ago a competent man was started out on a motorcycle to make an inspection and a detailed report of each patrol section in the state, showing the mileage under each patrolman, the character and condition of the road, the proficiency of the patrolman, the cost per mile, the tools and material needed, with instructions as to the work, recommendations as to the future, etc. These reports will be valuable in determining the comparative cost of oiling different sections, in showing up the good and bad work, the weak points in the system, and in placing the resident engineers and patrolmen in competition, etc.

During the latter part of 1913 a large amount of stone, chips, gravel and other repair material has been systematically stored along the roads in the state at convenient points, so that it is on hand when needed, and can be used more economically and advantageously than when ordered in a haphazard way.

Next year the floating gang system may be introduced in connection with the patrol system. A gang of experienced men, consisting of a foreman, with a traveling outfit, patching material, tools, etc., will go over a section of 100 to 150 miles of road in a season, doing the ordinary repairing and maintenance, besides unusual work, such as painting bridges, repairing culverts, remedying defective drainage, instructing patrolmen, etc. It is believed that the work can be done better and cheaper this way and that the patrol sections can be made much longer.

The following tabulation gives the increase in the patrol service:

Year.	Number of Patrolmen.	Miles. maintained.	Avge. number miles maintained per patrolman.
1910	2	...	...
1911	55	305	5.55
1912	75	374	4.98
1913	107	575	5.37

There were also employed from time to time, as necessary for extraordinary repairs, 150 assistants to the patrolmen in 1912 and 345 in 1913.

It may be explained that the number of patrolmen needed depends upon a variety of considerations, such as the type of road, the geological character of the country, the grades, the amount and kind of traffic, the width of the roads, etc.

When the whole road system is completed it will probably be wise to furnish each resident engineer with a steam roller at the center of his territory of two or three counties, together with an oiling equipment and other tools for prompt repair and emergency work.

Maintenance Costs by Years.—Table II shows the cost of oiling by years. Table III shows the cost of maintenance by the patrol system which has no immediate connection with the road oiling forces. The total cost was as follows:

1912	$340
1913	134

In 1912 there were 374 miles maintained, at a cost of $63,358.99, making an average of $169.10 per mile.

TABLE III. COST OF MAINTENANCE PER MILE (EXCLUSIVE OF OILING) BY YEARS, USING PATROL SYSTEM.

Item.	1912.	1913.
Patrolmen, helpers, teams, etc.	$ 93.40	$122.00
39,502 gals. of oil for repair work, 165 gals. per mile	8.00	....
98,561 gals. of oil for repair work, 172 gals. per mile	....	13.94
3,916 tons of stone and chips or grit for repair work, 10.5 tons per mile	13.30	....
9,918.25 tons of stone and chips or grit for repair work, 17.3 tons per mile	....	23.80
Bridge repairs	12.30	5.60
Maintenance on turnpikes	8.00	8.90
Traveling repair gangs	32.40	62.00
Patrolmen, tools, etc.	2.00	1.20
Totals	$169.40	$237.44

In 1913 there were 575 miles maintained, at a cost of $136,530.81, making an average of $237.41 per mile.

Construction Plant

Machines Devices Materials

Construction Plant—Machines, Devices and Materials.

A fairly comprehensive knowledge of the machines, devices and materials employed in constructing and equiping engineering works is essential to engineers and contractors. The contractor who bids closest, other things being equal, is the one who knows best the available machines for construction work—their cost, their capacity, their adaptability and the expense of operating them. The engineer who plans construction well must know well the appliances and materials which are available for special services. To possess this requisite knowledge both the contractor and the engineer must follow the news of improved and new machines and materials and appliances. This task is persistent because efficiency of equipment for doing work is increasing very rapidly. The number of machines and appliances devised for special tasks is increasing. More machines than ever before are being modified to utilize electric and oil engine power. Invention of devices for working and applying new materials of construction is more active than ever before.

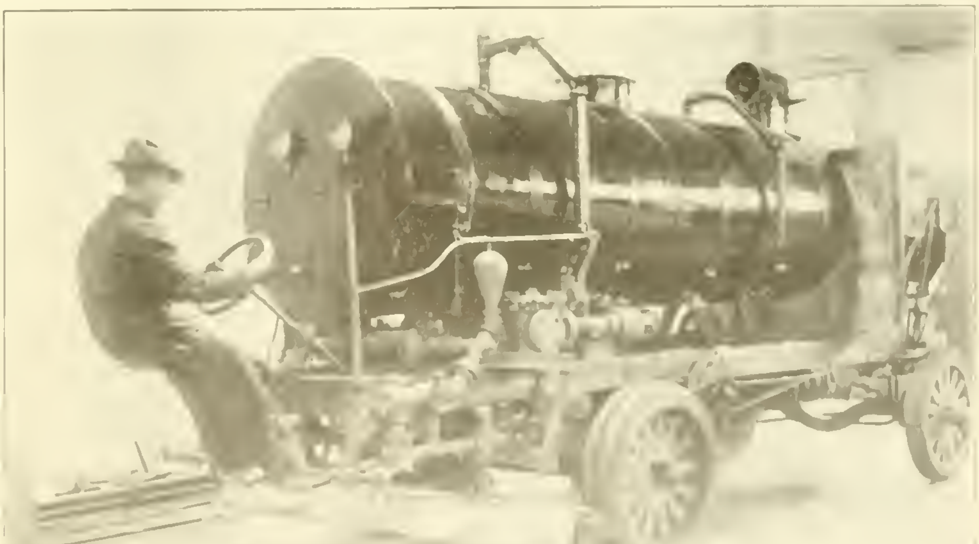
No enumeration and description of machines, devices and materials used in engineering construction will provide the contractor or engineer automatically with a well chosen plant, but it will place in his possession data essential to wise choice. The publishers of ENGINEERING AND CONTRACTING have therefore arranged to supply this service first by the preparation by an authoritative writer of a 700-page book on Construction Plant, and, second by the starting, with this issue, of a permanent section in ENGINEERING AND CONTRACTING, which will be devoted exclusively to Construction Plant—Machines, Devices and Materials The book presents as closely to date as is practicable in that form of publication, illustrations, descriptions, costs, capacities, operating charges and like data on all classes of construction plant in considerable use in engineering construction. This section of ENGINEERING AND CONTRACTING will continue the book by collecting and publishing the news as it develops of improved and new machines, devices and materials employed in engineering construction. This section is an innovation only in the respect that it assembles under one heading all construction plant articles. Previously such articles have been distributed through the reading pages, each being placed in the section covering the work for which the device or machine or material described was intended.

The planning of a plant for engineering construction whether it be by an engineer or by a contractor is an engineering problem. Like the choosing of permanent equipment and special materials for the engineering structure being built it calls in every instance for special study. The conditions, the requirements and the means available must be co-ordinated. This part of construction plant planning will not be discussed in this section but will be presented as in the past in connection with the sewer or bridge or tunnel or other structure being considered. Only the plant units will here be illustrated and described.

As in the choice of other articles for publication the selection of plant articles will be determined by the editor's opinion what for the reader is of most news value and general utility. No other basis of choice is practicable or justifiable. It is obvious too that the information presented must in considerable measure be that furnished by the manufacturers of the articles described. Except in matters of capacity, which is always an elusive quantity, these statements are rarely unreliable.

A Flexible Mechanical Distributor for Bituminous Material.

The requirements of a mechanical distributor for bitumen are that an even temperature and a uniform pressure shall be maintained, and the control of the distributing device shall be flexible to eliminate bare spots and overlaps and to control the quantity of material applied. The distributor illustrated herewith is manufactured by the Kinney Manufacturing Co., 102 Boylston St, Boston, Mass., in both motor driven and horse drawn types.



Kinney Combined Heater and Bitumen Distributor—Motor-Driven Type.

The motor driven machine is 19½ ft. in

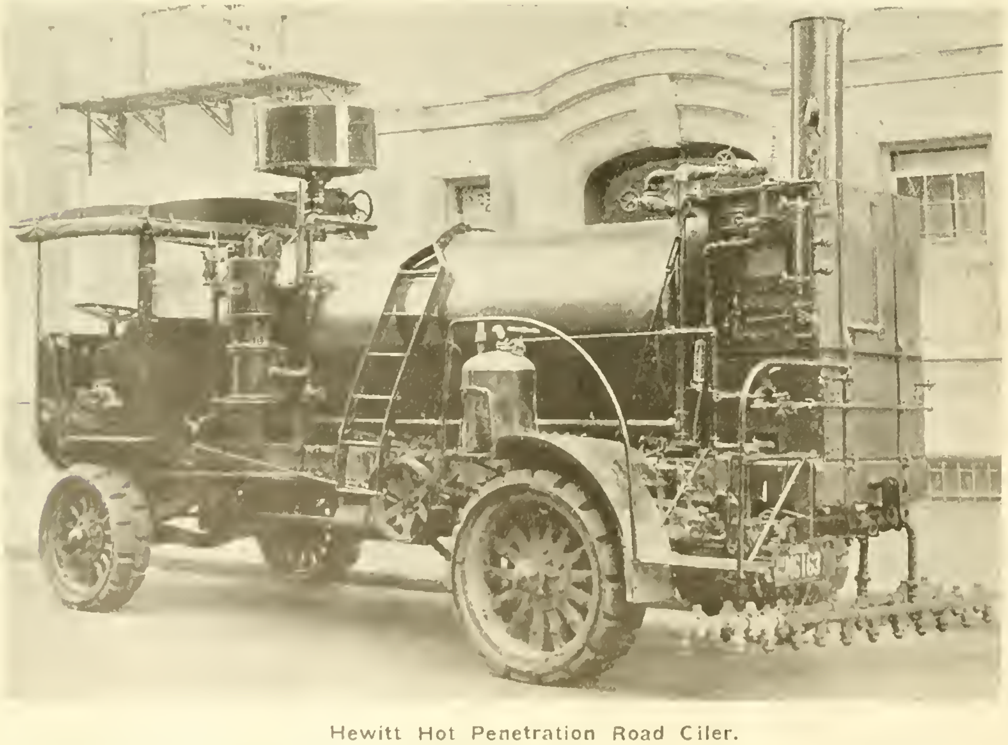
length of standard chassis and has a tread of 68 ins. and 72 ins. for the front and rear wheels, respectively. The bitumen is heated by tubes within the tank through which the heat derived from a kerosene oil burner passes. Pressure is maintained by means of a rotary pump which also agitates the bitumen in the tank. In addition to the distributing nozzles there is a hose connection on the side for patching and hand spraying. The nozzle distribution may be varied from 4 ft. to 12 ft. A means is provided by which the nozzle may be flushed with kerosene oil from the fuel tank at the end of the day's work. The frame on which the nozzles are mounted have a lateral motion controlled from the rear end. A sheet iron hood is provided to catch drippings. The normal road speed is 11½ miles per hour.

The horse drawn machine is operated by one man. The tank capacity ranges from 450 to 1,000 gals. and the machine is made with or without the heating device. The distributor may be obtained separately to attach to a sprinkler. The distributor and tank may also be used for water by adding a suitable attachment.

The Hewitt Hot Penetration Road Oiler.

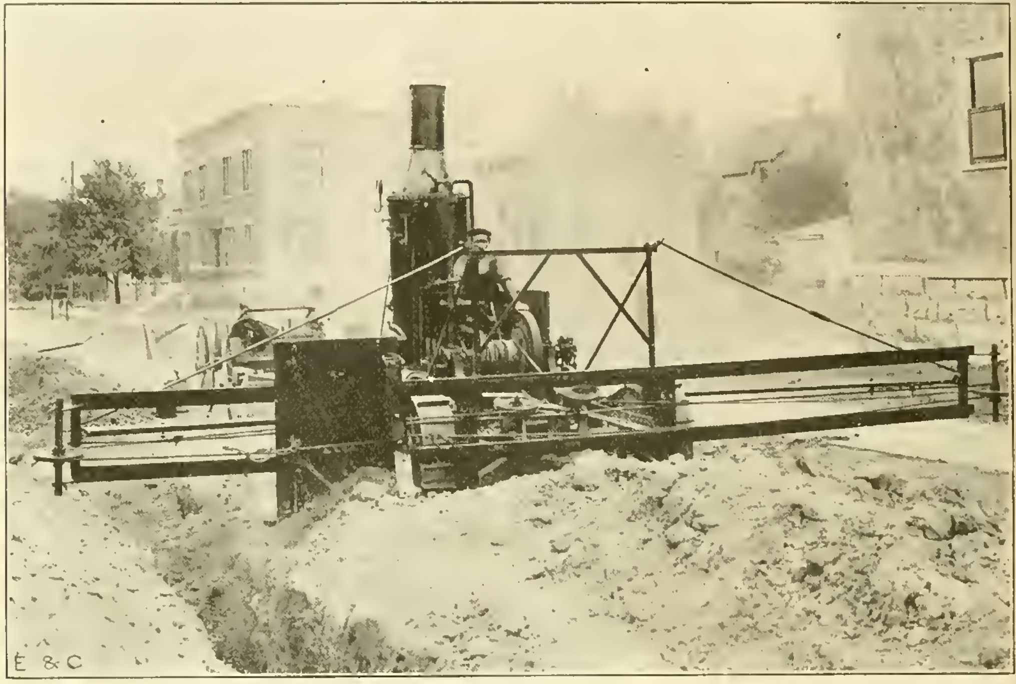
Machines for applying bitumen are changed and improved in details every year. Many of them are becoming as complete and flexible in operation as a locomotive. While the initial cost of equipment is, undoubtedly, increased the efficiency of the machine is often more than correspondingly improved.

The motor oiler illustrated is manufactured by the International Motor Co., West End Ave. and 65th St., New York, and has interesting features. The truck on which the tank is mounted has a capacity of 5½ tons with a good range of speed. The 750-gal. tank containing the binder is heated by steam coils the steam being generated in a flash boiler at the rear of the tank. Fuel oil is carried in a cylindrical tank on right hand side of the truck, another tank just below carrying water for the boiler. The air compressor is run by steam. An air primer is provided for starting the fire under the boiler. There is a straining device between the tank and the distributing nozzles which takes out matter that might tend to close or clog the nozzles. A hose may be attached to the nozzles for hand application. The range of distribution is from ¼ gal. to 2 gals. per square yard and is controlled from the operator's platform at the rear of the machine. This machine was developed for and is in use by the Standard Oil



Hewitt Hot Penetration Road Oiler.

A New Backfilling Machine.



View of Monahan Backfilling Machine in Operation.

The new machine for backfilling trenches illustrated herewith consists essentially of a 10-HP. engine and boiler suitably mounted and equipped to serve both for its own traction and as a single drum, reversing, hoisting engine; of a trackway attached to the end of the framework carrying the power equipment and extending at right angles to the direction of traction; and of a backfilling bucket or scraper which slides back and forth on this track.

The trackway consists of two Z-bars of 3x3x⅜ in. section 18 ins. apart and 24 ft. long. To the track side of the bucket, which is a heavy rectangular steel plate, there are attached the axles for four 8-in. wheels with 2-in. flanges, which roll upon the web of the Z-bars as rails as the bucket moves back and forth. The bucket is rollered against the Z-bars to take both the horizontal thrust and the uplifting thrust exerted when the machine is in operation. Two 3-in. x 3-in. angles are riveted vertically against this carriage plate. To one of these angles, depending on which end plate of the bucket is bolted. This plate is a flat rectangle except for the outer 6 ins. of its width, which is bent in at an angle of 45° toward the center of the bucket. A flat iron, ¾x3½ in. in section, known as the pulling rail, extends across the front of the bucket. It is riveted to the bent portion of the end of the bucket and bolted to the second 3x3-in. angle previously mentioned, which in turn is riveted to the carriage plate of the bucket. The bottom of the bucket, called the apron, swings on the hinge which connects it with the end plate. The apron is 18 ins. long by 24 ins. wide and, in operation, is tilted like a slip scraper is in filling. A chain attached to the apron hooks over the pulling rail previously mentioned and thereby gives the apron the inclination desired and holds it in that position. It will be seen that the tilt of this apron governs the depth of spoil left on the ground. The bucket moves horizontally only and always works, therefore, at the bottom of the spoil bank. Since the traction wheels of the rig move on the spoil left outside the trench the tilting of the apron can cause this spoil bank to take any slope desired, thus a manhole can readily be passed by the outfit. If the spoil bank shifts to the other side of the trench and the direction of traction is to be continued as before, the rear plate of the bucket and the pulling rail are unbolted from the angles on the carriage plate, the rear plate is turned end for end and rail and plate are again bolted to the angles.

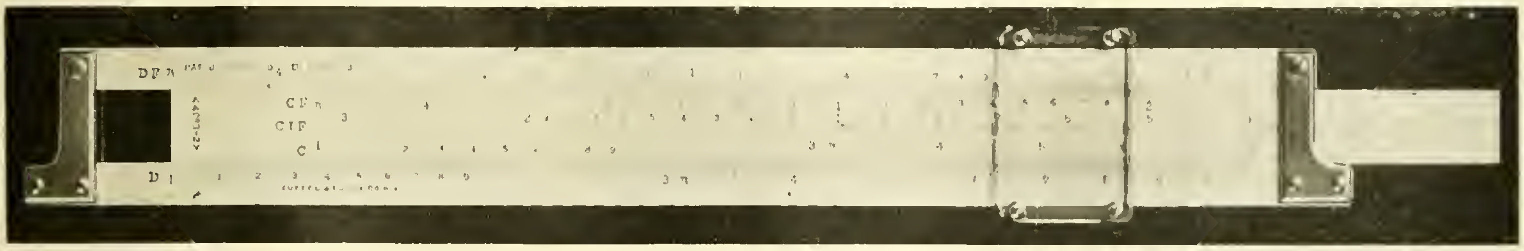
The bucket is actuated by a ⅜-in. wire rope which passes around the drum of the hoisting engine. This rope passes over four 12-in. sheaves, two of which are attached to the framework of the rig and properly tilted so that the rope winds on one side of the drum and unwinds from the other, and one sheave is attached to each end of the trackway as shown. The ends of the rope are fastened to the back of the bucket and to the pulling rail.

A ¾-in. bar runs midway between the Z-bars for their full length. To this bar are attached permanent trips which are struck by the bucket at the end of its stroke. One end of each trip is pivoted to the framework of the machine. When the bucket strikes the trip the latter swings on its pivot and moves the ¾-in. bar mentioned above. The end of a bell crank fits between two lugs at the center of this bar and as the bar moves endwise, first one way and then the other, the crank automatically reverses the engine through its connection with the reverse lever, and the bucket thereby changes its direction of motion. Adjustable trips can be placed where desired to shorten the length of the stroke.

make 15 strokes a minute on the average. The machine was invented and patented by Frank J. Monahan, a sewer contractor of Joliet, Ill., and inventor of the Monahan trench excavator, and is manufactured and marketed by William Heggie of the same city.

An Improved Form of Indicator for Slide Rules.

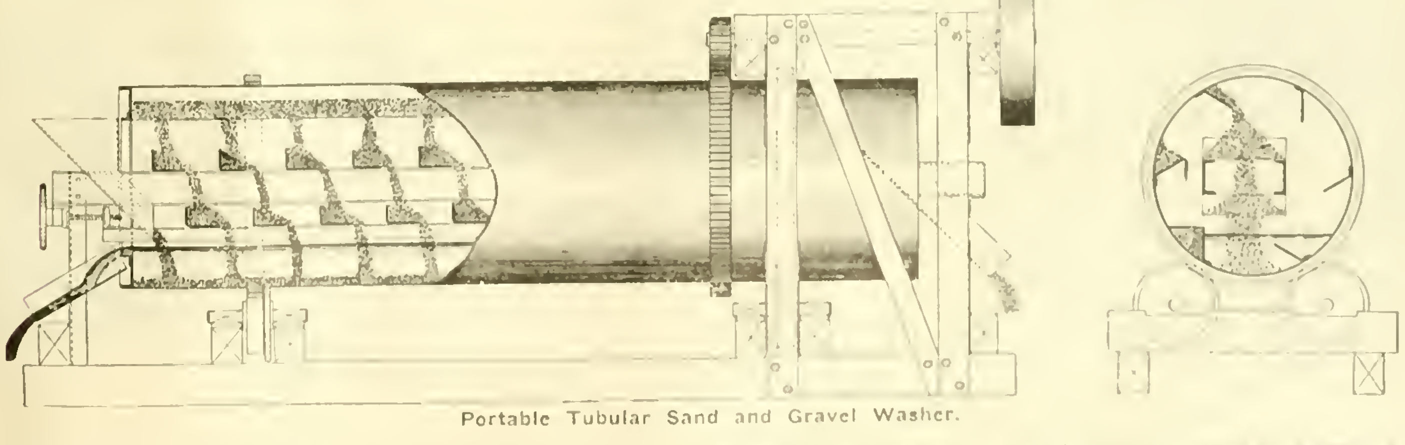
A new patented indicator, or runner, for slide rules, called the "Frameless," has just been perfected by the Keuffel & Esser Co. Every figure on the rule is clearly visible at all times, as there are no metal side pieces on the new indicator. Above is a view of the duplex form of the Keuffel and Esser rule, with the new indicator in place. The front of the runner is entirely of glass, which permits the figures to be easily read and thus removes one of the principal objections to the older forms. Hereafter, all adjustable slide rules of the Mannheim and Duplex types made by this company will be regularly equipped with the "Frameless" indicator.


Slide Rule with All-Glass Indicator.

A Portable Sand and Gravel Washer for Contractor's Use.

(Contributed.)

Preparation of concrete materials "on the job" often offers such opportunities for greater convenience and economy as to justify the contractor in having a thoroughly practical and effective washer as a part of his equipment. The accompanying drawing illustrates a portable tubular washer for contractors. Its operative principle is that of tumbling the materials about in a bath of water as they progress through the length of the cylinder. By repeated lifting and dropping onto intermediate shelves and into the water the lumps of material are broken up, all sticking foreign matter is scoured off, and the pebbles and sand delivered clean at the discharge end.


Portable Tubular Sand and Gravel Washer.

Lengthwise of the inside of the cylinder are lifting blades set in angular position, as shown by the sectional view. As the cylinder rotates, these blades carry up materials and water, discharging them onto a double series of stationary angle-shaped shelves. The shelves retain certain quantities of the materials, to assist the scouring action on the falling materials. The presence of the retained materials also acts to prevent wear of the shelves themselves. To simplify the drawing and make clear the action, the fall of material from the lifting plates direct to the lower shelves is not shown. The material striking the higher shelves falls next to the lower ones and thence drops into the water below. The quantity of water raised with the materials by the lifting blades can be increased or decreased by varying the speed of rotation of the cylinder.

The materials progress through the length of the cylinder by the forward movement caused by the arrangement of the shelves. Regulation of the rate of forward progress—and therefore the time required for passage through—is effected by movement of the lower series of shelves. These are movable lengthwise of the cylinder, by means of the adjusting screw shown in the drawing. The lower shelves may be set directly beneath the upper ones, and for this position the progress of the materials is slowest. When set as far forward as will permit them just to catch the fall from the upper shelves, the rate of progress is fastest. With this easy adjustment for the rate of progress, the washer may be made to suit exactly the peculiar requirements of any materials, or any variation in the materials. Difficult materials may be washed very slowly; easier materials may progress rapidly through.

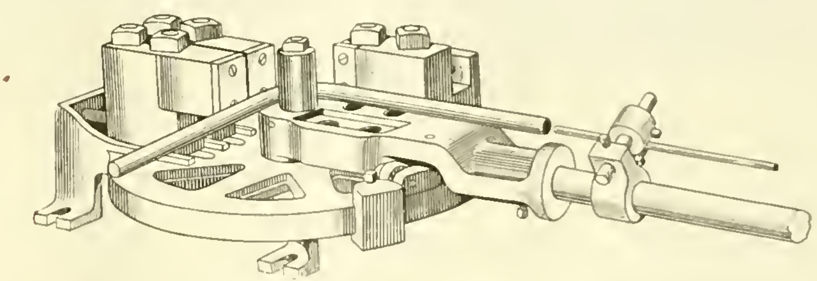
The direction of flow of the water is opposite to that of the materials, the water entering at the discharge end (on the right in the drawing) and flowing out at the feed end. In this way the washed materials as they tail out are rinsed clean by the fresh water, and the primary work of breaking up the lumps is done by the dirty water at the feed end.

By the very nature of the operative principle in this washer the quantity of water required is a positive minimum. Only so much water is necessary as will flush away the impurities at the discharge end. Continued tumbling of the materials loosens and scours off all the clinging dirt regardless of how dirty the water may be, and final cleaning is assured by the rinsing inflow of clean water at the delivery end. No greater economy of water could be possible.

Size separation may be provided for in very simple manner by attachment of a conical screen extension at the delivery end of the washing cylinder. For independent screening—and better separation if several sizes are wanted—the discharge from the washer may be passed to any suitable set of independent screens.

The wide range of application for these washers calls for their construction in several sizes. They are now built regularly in diameters from 30 to 60 ins., with lengths of 8 to 20 ft. Special proportions can be arranged to meet any peculiar conditions to which a regular size may not be suited. The cylinder is made of heavy steel, encircled by steel tires which run on four flanged wheels with chilled treads. Driving gears are extra heavy and are so placed as to avoid contact with the materials. The drawing shows straight drive, angle drive may be furnished at no extra cost.

This washer is manufactured by the Raymond W. Dull Co., 1910 Conway Building, Chicago, Ill.


Device for Bending Rails or Bars for Reinforcing concrete.

Device for Bending Steel Bars or Rods for Reinforcing Concrete.

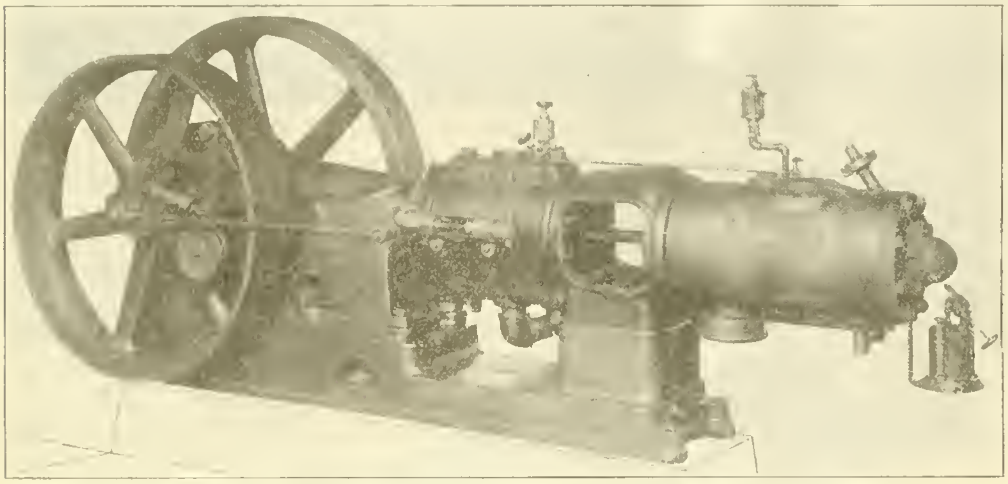
(Staff Article.)

The above sketch shows a bar bending tool for bending bars or rods of any shape ordinarily used for reinforcing concrete. This tool will bend bars to V-shape or to any angle up to 180° with a radius at the point of bending equal to the radius of the forming pin. Different sizes of forming pins can be used and a change of size can be quickly made. When rectangular bends are wanted a rectangular die is substituted for the pin. With this die bends to 120° can be made. The machine has capacity to bend cold ⅜x3-in. flat bars, 1-in. round bars, ⅞-in. square bars and ¾-in. square twisted. Bending hot it has capacity to bend, respectively ⅝x3-in., 1¾-in., 1½-in. and 1½-in. bars. This bending tool is made by the Wallace Supply Co., 412 Orleans St., Chicago, Ill.

An Oil Engine Driven Air Compressor.

(Contributed.)

The increased use of low grade oil fuel for power purposes has led to the design by the Ingersoll-Rand Co., of the oil engine driven air compressor illustrated. This is of the direct-connected straight line type and somewhat resembles in this respect, as well as in the design of the air end, the company's standard line of small compressors. The main frame is designed for a splash system of lubrication; it is of the wholly enclosed type and provided with removable covers.



Oil Engine Driven Air Compressor.

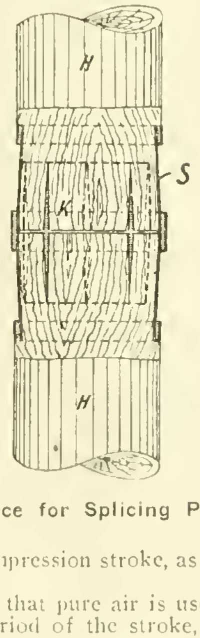
The feature of greatest interest in this machine is the design of the driving end. This, as can be seen from the illustration, consists of a single oil engine cylinder set behind the air cylinder and directly connected by means of an extended piston rod, to the air piston. It follows in general design a type known as the hot bulb engine, which is really a development of the Diesel engine, and combines a high thermal efficiency with great simplicity of construction and far less complication of details of design; there being an entire absence of auxiliary air compressors, etc., this, combined with a lower working pressure, makes an ideal type for compressed air service.

Dumped Position of Tilting Wagon Loader

The power cylinder is of the single acting, two cycle type. It is water-jacketed and provided with an efficient system of lubrication and is of sturdy design, it is supported by a heavy distance piece reaching to the foundation and bolted to the air cylinder. It is fitted with a torch for heating the ignition bulb preliminary to starting. After the compressor is under way this torch is dispensed with.

The fuel is automatically injected into the combustion chamber by means of a small pump on the side of the frame, operated by the main shaft. It enters in the form of a finely atomized spray and is immediately ignited by the hot bulb, dispensing entirely with electric sparking devices, batteries, etc. The stroke of the fuel pump is regulated by a centrifugal governor located in the flywheel, thus regulating the amount of fuel injected into the cylinder in proportion to the load. This is supplemented by a regulating device on the intake to the air cylinder of standard design.

The operation of this machine is accompanied by none of the losses common to the average two-cycle gasoline engine, in which part of the incoming charge follows the exhaust gases through the outlet ports and is wasted. This is due to the fact that the fuel is not vaporized by an outside agency and introduced with the air used for scavenging, but is injected directly into the cylinder at the end of the compression stroke, as already mentioned.



Device for Splicing Piles.

This means that pure air is used during the scavenging period of the stroke, consequently the inlet and outlet ports can be so arranged that more scavenging is afforded without any loss of fuel. The absence of carburetor, with its needle valves, springs and delicate adjustments which have to be constantly changed to suit atmospheric conditions, is an advantage which cannot be over estimated.

A feature of this engine is the introduction of a small quantity of the water from the cylinder jacket into the combustion space. This water performs the function of regulating the temperature in the cylinder, thereby preventing an undue rise in temperature of the piston, etc., causing disassociation of the fuel. It reduces the maximum pressure in the cylinder, at the same time slightly increasing the mean effective pressure, making a smooth running and highly economical machine. The amount of water injected is regulated according to the load on the compressor.

The whole machine is compact and self-contained. It is at present made in but one size with an actual capacity when running at 325 R. P. M. of 66 cu. ft. of free air at 100 lbs. pressure and 73 cu. ft. at 80 lbs. pressure. The fuel consumption at this speed, and under average operating conditions, is about 2.2 gals. of kerosene per hour. It is adapted to run on either kerosene, fuel oil or distillate. Its weight complete is 3,000 lbs. and floor space 8 ft. 10 ins. x 2 ft. 5 ins.

Device for Splicing Timber Piles.

An ingenious device has been patented by Mr. Heimbach, of Bregenz, Austria, for combining two wooden piles into a rigid pile. By means of this connection a pile of any desired length can be obtained. The cut shows two wooden piles, *H*, joined together. A tube, *S*, with two conical ends and reinforced with three rings, as shown, is driven over the lower pile so that one-half of its length projects above this pile. A double annular steel wedge, *K*, having radial ribs is placed over the top of the lower pile and the upper pile is then placed over this wedge. The piles are then driven together, which forces the wedge into the ends of both piles and thus completely fills the double conical tube *S*. A modified form of this device permits a wooden and a concrete pile to be spliced.

A Tilting Car-Side Wagon Loader.

The loader illustrated was invented and is manufactured by A. P. Lee, proprietor of the Arrow Motor Cartage Co. of Chicago, and consists of a set of folding standards of any length desired supporting a tilting bucket of 1½, 2, 2½ or 3 cu. yds. capacity. The standards are on wheels and when in position are firmly braced. The bucket is supported upon angles attached to the standards and is not balanced but is held firmly in the illustrated loading position by a wire cable which passes over a winch for returning the bucket to the loading position and which is operated from the ground. Dumping is accomplished by one man by means of levers operated from the car.

Loading Position of Tilting Wagon Loader.

An important feature of this bucket is the small clearance required for the dumping operation. The outer lip of the bucket in the dumped position is 14 ins. lower than the inner lip in the loading position, which is the clearance required for operation.

In operation these loaders are usually used three to a car, although this is not necessary. They are also made, without standards, to be attached to the car side.

Engineering and Contracting

Devoted to the Economics of Civil Engineering Design
and to Methods and Cost of Construction

Volume XLI. CHICAGO, ILL., JUNE 24, 1914. Number 2

The Efficiency of Irrigation Works.

The carefully prepared article on irrigation water losses which is published in this issue deserves the most serious attention. Throughout the arid region of the United States there is as a rule more good land than the water available will adequately irrigate. The profit of irrigation work lies in the increased value of the land served with water. Waste of water is by both counts an economic crime. The waste of water is enormous in most irrigation works. Of the water entering irrigation storage reservoirs careful study indicates (E. & C., Nov. 6, 1912) that from 10 to 60 per cent evaporates and seeps out and never enters the outlet canal. Of the water entering the main canals from 17 to 45 per cent is lost by evaporation and seepage and never enters the distributing laterals. Of the water entering the distribution canals from 30 to 50 per cent is lost by seepage and evaporation before the water gets to the farmer. Of the water taken by the farmer 10 to 15 per cent is wasted by surface run-off. These are all primary counts, all economic niceties of irrigation water utilization being unconsidered. No engineer needs to have the significance of such figures explained. If, with from 10 to 90 per cent of the available water supply wasted, we can irrigate the acreage that we now do, how many acres could we irrigate if a quarter or a half or three-quarters of the water now wasted were not wasted? This is not an idle query. As great saving and often greater saving are entirely possible. Simply lining the canals will often accomplish these savings. We ask our readers to study the article published in this issue and then to read the article by Mr. E. G. Hopson, published in our issue of Nov. 6, 1912.

Bond Between Concrete and Steel.

Designers of reinforced concrete structures in general appreciate the necessity of providing sufficient bond area to develop the strength of the reinforcement, although some engineers have exaggerated ideas of the value of various forms of reinforcing bars. A considerable amount of valuable data on the bond between concrete and steel is given in Bulletin No. 71, University of Illinois Engineering Experiment Station, the conclusions drawn from the series of tests recorded being given in this issue. Some of these conclusions are of special interest and deserve careful consideration as they are the result both of pull-out tests and beam tests.

It is stated that the bond between concrete and steel may be divided into two principal elements, adhesive resistance and sliding resistance, the tests indicating that the former is much the more important element of bond resistance. It was found that, in the pull-out tests of deformed bars, the adhesive resistance must be destroyed, the usual sliding resistance largely overcome, and the concrete ahead of the projections must undergo an appreciable compressive deformation before the projections on a deformed bar become effective in taking bond stress. The tests indicate that the projections do not materially assist in resisting a force tending to withdraw the bar until a slip has occurred approximately equal to that which corresponds to the maximum sliding resistance of plain bars. The maximum bond stresses for the deformed bars corresponded to an end slip of 0.1 in., and as evidence of splitting of blocks not reinforced was found at end slips of 0.02 to 0.05 in., it is evident that members must be reinforced against splitting if increased values for deformed bars are allowed. The bearing stresses between the projections and the concrete were computed to be from 5,800 to 14,000 lbs. per square inch at the highest bond stresses considered in the tests.

The tests indicate that the projections of deformed bars should present bearing faces as nearly as possible at right angles to the bar to avoid wedging action, and that to develop a high bond resistance a certain amount of metal must be used in the projections which probably will not be available for taking tensile stress. The tests of 1-in. twisted square bars gave a bond resistance per unit of surface at an end slip of 0.001 in. of only 88 per cent of that for plain round bars. Following an end slip of about 0.01 in. the twisted square bars showed a decided decrease in bond resistance, and some of the bars had to be withdrawn 2 or 3 ins., in blocks reinforced against bursting, before the highest resistance was reached. It is obvious that increases in bond resistance which do not occur until there is a large slit have little practical significance.

Tests of plain round bars anchored by means of nuts and washers showed that the anchorage did not become effective until the bar had slipped an amount corresponding to the maximum bond resistance of plain bars. With a further movement, the bond resistance was high, but was accompanied by excessive bearing stresses on the concrete. Specimens made out-doors in freezing weather, where they probably froze and thawed several times during the period of setting and hardening, were almost devoid of bond strength.

In the beam tests, slip of bar was first observed in the middle region of the span at loads producing a tensile stress in the steel of about 6,000 lbs. per square inch, the slip of bar being probably partly responsible for the opening of outer cracks, since slipping was observed in the outer thirds of the beam before the cracks became visible. It was found that, in beams reinforced with plain bars, the end slip began at 67 per cent of the maximum bond resistance; for corrugated bars this percentage was 51; and for twisted square bars end slip began at 66 per cent of the maximum bond resistance. The bond resistance in beams reinforced with plain square bars, computed on the superficial area of the bar, was only about 75 per cent of that for similar beams reinforced with plain round bars of similar size. At small slips the twisted square bars gave values about 85 per cent of those for plain rounds, while at the maximum load this percentage was 90. In beams reinforced with 1⅛-in. corrugated rounds, slip was observed at about the same bond stress as in plain bars of comparable size, while at the maximum load the corrugated bars gave values about 30 per cent higher than the plain rounds. The reinforced concrete beam tests showed that very small amounts of slip at the ends of the bar represented critical conditions of bond stress. For beams failing in bond the load at an end slip of 0.001 in. was 89 to 94 per cent of the maximum load found in beams reinforced with plain bars, and 79 per cent of the maximum load for similar beams reinforced with corrugated bars. As soon as slip of bar became general other conditions were introduced which soon caused the failure of the beam.

It was concluded from the tests that a working bond stress equal to 4 per cent of the compressive strength of concrete, tested in the form of 8x16-in cylinders at the age of 28 days, is as high as should be used. This is equivalent to about one-third of that causing
first slip of bar and about one-fifth of th
maximum bond resistance of plain round ba
as determined from pull-out tests. The u
of deformed bars of proper design may be e
pected to guard against local deficiencies
bond resistance due to poor workmanship, a
their presence may properly be considered
an additional safeguard against ultimate fa
ure, but it does not seem wise to place t
working bond stress for deformed bars high
than that used for plain bars.

The Cost of Building Roads Wit
Convict Labor.

There is, undoubtedly, a great need for d
tailed and accurate costs of equipping, ope
ating and maintaining convict labor ro
crews together with records of the work pe
formed by them. These data need to be co
lected over an extended period of time, s
four or five years, in order that the figur
derived as to the cost of maintenance m
have some degree of accuracy. The prese
dearth of accurate data on this subject is
be regretted, since there is a marked te
dency on the part of different states to empl
more labor of this sort in their road constru
tion work, and accurate information wou
be of great value in securing a well reg
lated organization.

The majority, by far, of the convicts e
ployed in road construction are used by cou
ties of various states in the southern porti
of the country. For the most part these c
victs are negroes, and it has been found t
this type of labor is efficient and quite tra
table when properly handled. While th
are many reasons to believe that the w
accomplished has been both satisfactory
quality and economical in cost, as a rule, v
few of these counties keep any cost reco
from which accurate comparisons may
made.

There are probably several reasons for t
An important one in small counties is t
the number of men employed (usually fr
15 to 30) is not large enough to warrant
ploying a superintendent with the all aro
ability and training necessary to lay out
supervise the work and control and care
the men. The type of work accomplis
(grading and maintenance of earth roads
the most part) is such that much engineer
supervision is unnecessary. In the lar
counties where several gangs are emplo
and in sections where the convicts are su
vised by the state there is no excuse for
condition of affairs, and accurate cost
should by all means be kept.

In other sections, with the possible ex
tion of the states of Washington and Ore
road construction with convict labor is lar
in an experimental stage. The fragmen
records available indicate that most of the
periments have been successful. We bel
that in sections of nearly every state in w
certain types of road may be built the us
convicts is desirable from both an econon
and a humanitarian standpoint, and in o
sections of the same state or on diffe
types of construction their use is not a
able. So much available data indicate.
complete data might lead to different co
sions.

In this issue of Engineering & Contr
ing, an article is published illustrating
methods of securing costs which are use
the Alabama Highway Commission.
system is worthy of close study by those
have to do with the operation of convict
in the South. In its main features it ca

adopted by a county without change in organization and its use involves little additional work on the part of officials—merely a systematization of their former work. Briefly, its advantages are that it is simple and flexible to meet widely varying conditions and very little computation is done by the foreman. The last feature is quite important. The writer has found it impracticable to expect much of a convict construction foreman in the way of computing cost data. The pocket memorandum used in Alabama is, however, an aid to the foreman in preparing his payrolls and from the data given satisfactory, if not absolutely accurate, costs may be derived. The records of costs of maintenance of output are valuable and if kept consistently the figures from Alabama will in a few years be of utility to other states in the South.

We wish to urge road officials in the various counties of the states in which the convict labor system is well established to keep data of this nature. In doing this they will perform a service to engineers in general and it will undoubtedly aid them materially in introducing economies in their own work.

Growing Sentiment Favoring Meterage in Chicago.

There is a change taking place in Chicago in the attitude of citizens toward the installation of water meters. A short time ago, in a communication to the city council, the mayor stated that the waste of city water is so great as to be practically a municipal crime. The mayor favors an extension of the meter system in this city and the commissioner of public works, the city engineer and the superintendent of water concur in this attitude toward meterage. The city council, however, is still inclined, it seems, to look upon the water meter as a device conceived primarily for the purpose of separating the citizen from his money. That this view is opposite the truth, especially for conditions as they exist in Chicago, is the conviction of the above named officials. The problem of educating the city council to a realizing sense of the economic advantage, to the householder, of the sale of water on the meter basis, is a problem for the Chicago newspapers. Consequently, it is gratifying to note that the leading daily papers are now, for the first time, advocating meterage. Formerly they subscribed to the doctrine that water should be as free as air. With the lake in the city's front yard the newspaper men could not understand why a limitation should be placed on the waste and improper use of water. These men have now discovered that it costs money to distribute the water and they are, consequently, more tolerant of the proposition that a man should pay for the service he receives in proportion as he is served.

Of course there have been other dissenting voices than those heard in the council chamber. All of the old objections to metering water have been brought forth as fresh discoveries. Thus we hear that the installation of meters will deprive poor folks of sufficient water for their actual needs. Sometimes we wonder where that thought originated and many times we marvel at its durability. It is the hardiest bit of misinformation of which we have knowledge.

In addition to the well-seasoned objections to meterage there have been some brand new ones. That such new objections would be raised might have been anticipated for a class of people, not without experience in raising objections in other lines, have been admitted to the rights of citizenship since meterage was last agitated in Chicago. Some of the objections raised before council committee by the women deserve record here. One feared that "no meter could measure the service with the water as thick as it is." Another stated that in the event of metering all services the pavements would have to be torn up at intervals of 25 ft. on each side of the street.

Despite the foregoing objections the coming of meterage in Chicago is inevitable because the meter furnishes the only way of selling water on an equitable basis. We hope that the daily newspapers will take the lead in this constructive work and make it a real issue at coming aldermanic elections.

Buildings

Architectural Features of the New Station and Office Building and Other Terminal Improvements of the Michigan Central Railroad Co., at Detroit, Mich.

Contributed by F. A. Pruitt, Detroit, Mich.

I.

The Michigan Central Railroad Co. has recently completed, at a cost of about $2,500,000, its new station and office building at Detroit, Mich., this improvement being the last step in the extensive plan worked out by this company. The entire project includes the tunnels under the Detroit River, the train sheds, yards and equipment, grade separation and bridges, coach yards, service building, warming building, hump yard, custom house, inspection sheds, wheel pit, signal towers, yard electrification, team tracks, etc. With this work completed the company now has at this point one of the most complete and fully equipped plants for the handling of freight and passengers to be found in this country. In this issue we shall treat, principally, of the layout and architectural features of this improvement, and in a succeeding issue shall cover the structural features of the work.

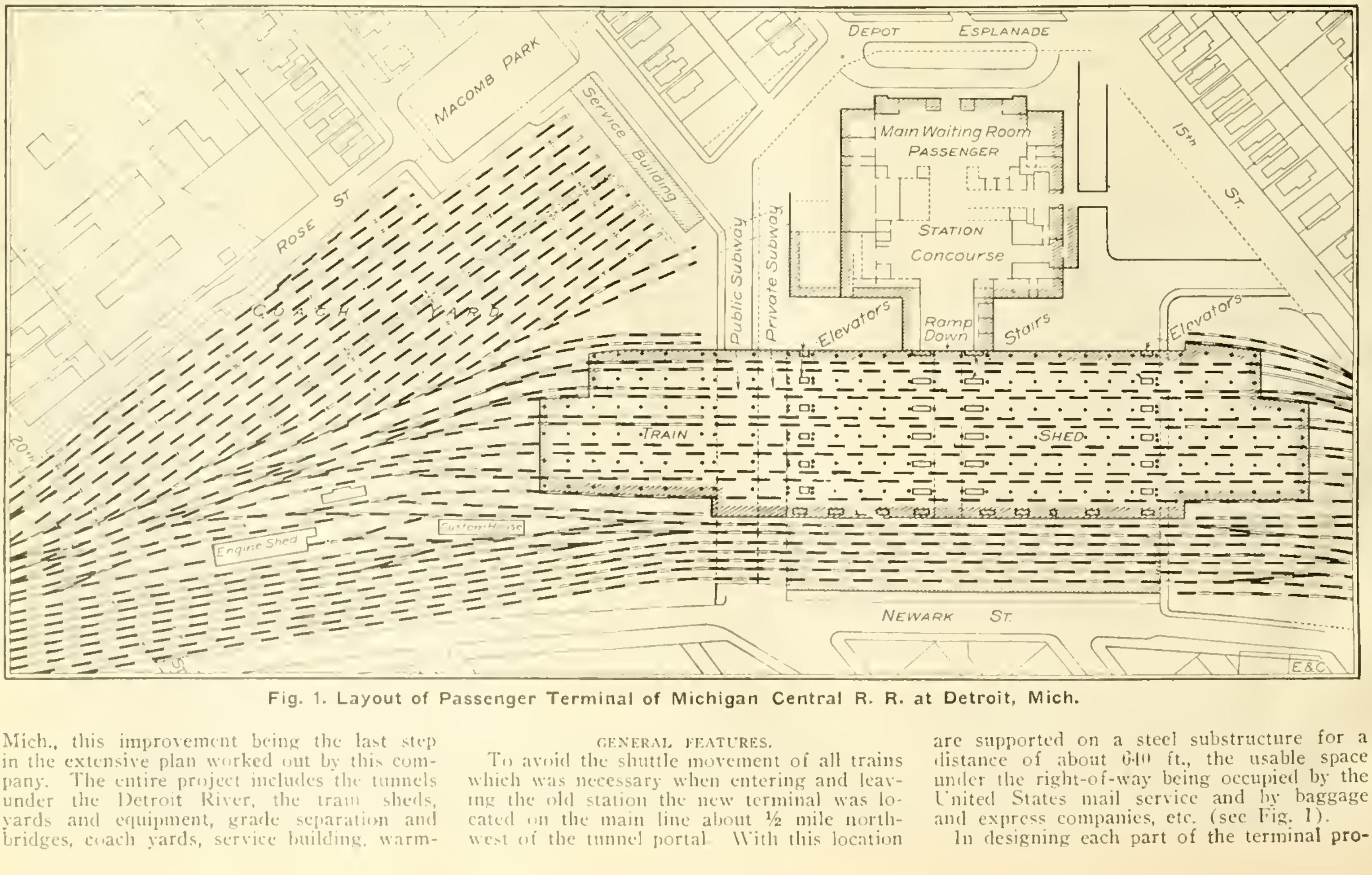


Fig. 1. Layout of Passenger Terminal of Michigan Central R. R. at Detroit, Mich.

GENERAL FEATURES.

To avoid the shuttle movement of all trains which was necessary when entering and leaving the old station the new terminal was located on the main line about ½ mile northwest of the tunnel portal. With this location and with the cross-overs in the train shed in use about 30 minutes can be saved in making up the trains and getting them across the river, from that required to operate to and from the Third St. Station.

The general scheme is that of a through station layout with 10 passenger tracks and 1 express track under the cover of the train shed. There are 7 through freight tracks in the open south of the train shed and parallel to it at the same elevation. All of the tracks are supported on a steel substructure for a distance of about 640 ft., the usable space under the right-of-way being occupied by the United States mail service and by baggage and express companies, etc. (see Fig. 1).

In designing each part of the terminal provision has been made for a considerable increase in traffic above that now required by the Michigan Central, in order that other roads in the city may be accommodated as tenants at any time they so desire. There is a possibility that the Grand Trunk and Lake Shore may later use this station. The Michigan Central alone handles, on an average, 5,000 passengers daily, with an estimated maximum of 9,000 on special occasions. The old station accommodated 75 trains, with a total daily capacity of 544 cars.

Location.—The station is located a little more than 1½ miles west of the City Hall and about the same distance from the old Third St. Station. It is about 800 ft. south of Michigan Ave., between 15th and 17th Sts. on Depot Esplanade, a new street and parkway, which was opened by the separation of grades (see Fig. 1).

Approaches.—All of the approaches have been designed and the grades separated so as to give the most efficient and easy access to the station and train shed, both for the public and for the tenants using the rooms under the tracks.

At the east end of the station there is a driveway about 40 ft. wide, sloping from 15th St. downward on a 5 per cent grade under the covered bridge which connects the station and the car shelter. This driveway gives access to the basement entrances leading to the basement elevator hall and to the machinery room which is directly under the main concourse floor. The driveway and a similar approach, which is perpendicular to and just south of the car shelter, merge into a large court about 78 ft. wide and 240 ft. long. This gives ready access to the mail department, which occupies the space under the tracks adjacent to this court.

Just north of the station and its surrounding sidewalk there is a curved driveway about 25 ft. wide leading from 15th St. around the granite balustrade to 16th St. Between this driveway and the Depot Esplanade there is an ornamental grass plot with blue flag stone landing and granite balustrade and benches.

On the west side and parallel to the station there is a 10-ft. driveway leading up to the carriage entrance which is under a low roof outside of the main office building. West of this driveway and separated from it by a granite balustrade is a driveway on about a 3 per cent down grade, which opens into a large court similar to the one on the opposite side of the building. This court is used by the baggage wagons, the baggage room being directly opposite under the tracks. Further west the Depot Esplanade and 17th St. merge into a public subway leading under the tracks and connecting with 17th and Newark Sts. on the south side of the train shed. Just east of this public subway and parallel to it there is a 40-ft. private driveway along the west end of the baggage and express rooms, which is used by baggage and express exclusively.

South of the train shed and parallel to it there is a driveway under the express companies' offices, which is used entirely by the latter.

THE MAIN BUILDING.

The lower story of the main building, which includes all of the station facilities, has a frontage of 345 ft. and a depth of 266 ft. These dimensions are maintained up to the top of the waiting room and concourse roofs. Above this level the office building, which is in the shape of the letter "I," rises to a height of approximately 240 ft. from the curb to the top of the penthouses. Figure 5 shows a front elevation of the station and office building and indicates the type of construction used.

From a structural standpoint the building may be described as consisting of three separate parts. Along the Depot Esplanade the general waiting room, with its adjoining womens' room, reading room and tea room, has a depth of 101 ft. and a height of about 63 ft. to the Guastavino arch ceiling. Directly to the south of the waiting room the building rises 15 stories above the high arches of the arcade ceiling. The "stem" of this part of the building is 51 ft. wide by 211 ft. long. At each end of this narrow stem there is a bay 52 ft. wide by 106 ft. long which completes the I-shaped outline of the office portion.

The concourse with its entrance for pedestrians on the east end and cab entrance on the west is located south of and parallel with the office portion. This room is 78 ft. wide, with a flat copper ceiling about 11 ft. above the street level. Leading south from this concourse there is a 76-ft. ramp, which grades down on a 7 per cent slope into the passenger subway under the tracks of the train shed.

The high portion of the building at the pipe loft and storage floor levels is separated from the waiting room and concourse by 13-ft. courts with hip-and-valley tile floors. These courts provide light for all offices on these levels, which would otherwise be shut off by the concourse and waiting room roofs.

Type of Construction.—The entire building is of fireproof construction, the steel skeleton framework being covered with stone, brick terra cotta or concrete.

The foundation is one of the first of its kind ever attempted for a building of this magnitude, a reinforced concrete mat being used under the entire building. The original design provided for caissons, but the experience of the Detroit River Co. in the construction of its tunnels showed that the blue clay had a tendency to flow under heavy pressure and the borings showed that the average distance to rock was 110 ft. These conditions together with the fact that there was a possibility of striking a sulphur vein, such as was found in constructing the Ford building, lead to a very exhaustive series of tests on the bearing power of clay and of several types of concrete piles. This series of tests indicated that piles were inadequate, and a reinforced concrete mat was decided upon. This ma has a thickness of 42 ins. under the high office portion and 30 ins. under the waiting room and concourse. As the type of foundation was changed after some of the columns were fabricated it was necessary to build up piers from the mat for some of the columns.

From the street grade up to the sills of the large waiting room windows which conform to grade the outside walls are of pink granite. From this elevation to the window sills of the first office floor the walls are of dark blue Bedford limestone; and from this level up to the 12th office floor a light-colored pressed brick, which harmonizes with the stone, was used.

At the 12th office floor the building sets in about 2 ft., thus providing space for the columns and festoon spandrels which are constructed of light gray terra cotta. The terr cotta construction continues up to and includes the massive overhanging cornice. Th entire main roof of the office building, a well as that of the reading room, tea room women's room and the flat roof over th offices of the low building, has a pitch-and-gravel roofing. A copper roofing with standing seams is used for the waiting room, concourse, cab stand and car shelter.

The car shelter at the east end of th building is reached from the main arcade o

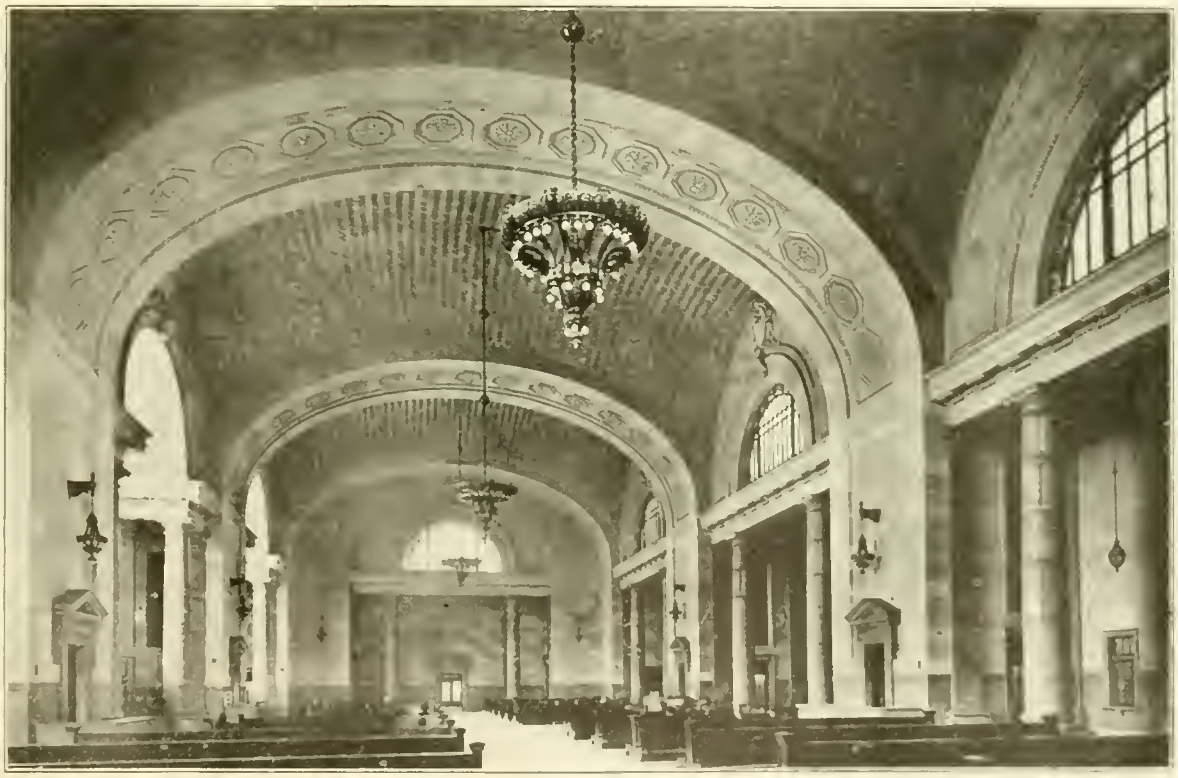


Fig. 2. View of East Portion of Main Waiting Room of Detroit Station, Showing Interior Finish.

the vestibule at the east end of the concourse by means of a covered bridge which is 21 ft. 9 ins. wide and which extends over the inclined driveway along the east side of the building. This shelter is adjacent to the trolley car loop, it is 17 ft. 9 ins. wide and about 300 ft. long, and is built on a radius of 60 ft. The framework of the shelter is of ornamental steel with the street side entirely enclosed with metal sash windows. A wood floor was placed in this shelter temporarily until the back fill settles to its final position, at which time it is intended to replace it with reinforced concrete slab construction.

QUANTITIES OF MATERIALS.

The terminal building contains 7,000 tons of structural steel, 125,000 cu. ft. of stone; 1,500,000 face brick, and 7,000,000 common brick. The foundations and side walls required about 20,000 cu. yds. of concrete and about 500 tons of reinforcing bars. The concrete fireproofing and the reinforced concrete floors required about 5,000 cu. yds., in addition to that used in the train shed and other outlying buildings. The train shed superstructure required 3,500 tons and the substructure about 9,000 tons of structural steel. The station proper has a volume of about 11,000,000 cu. ft.

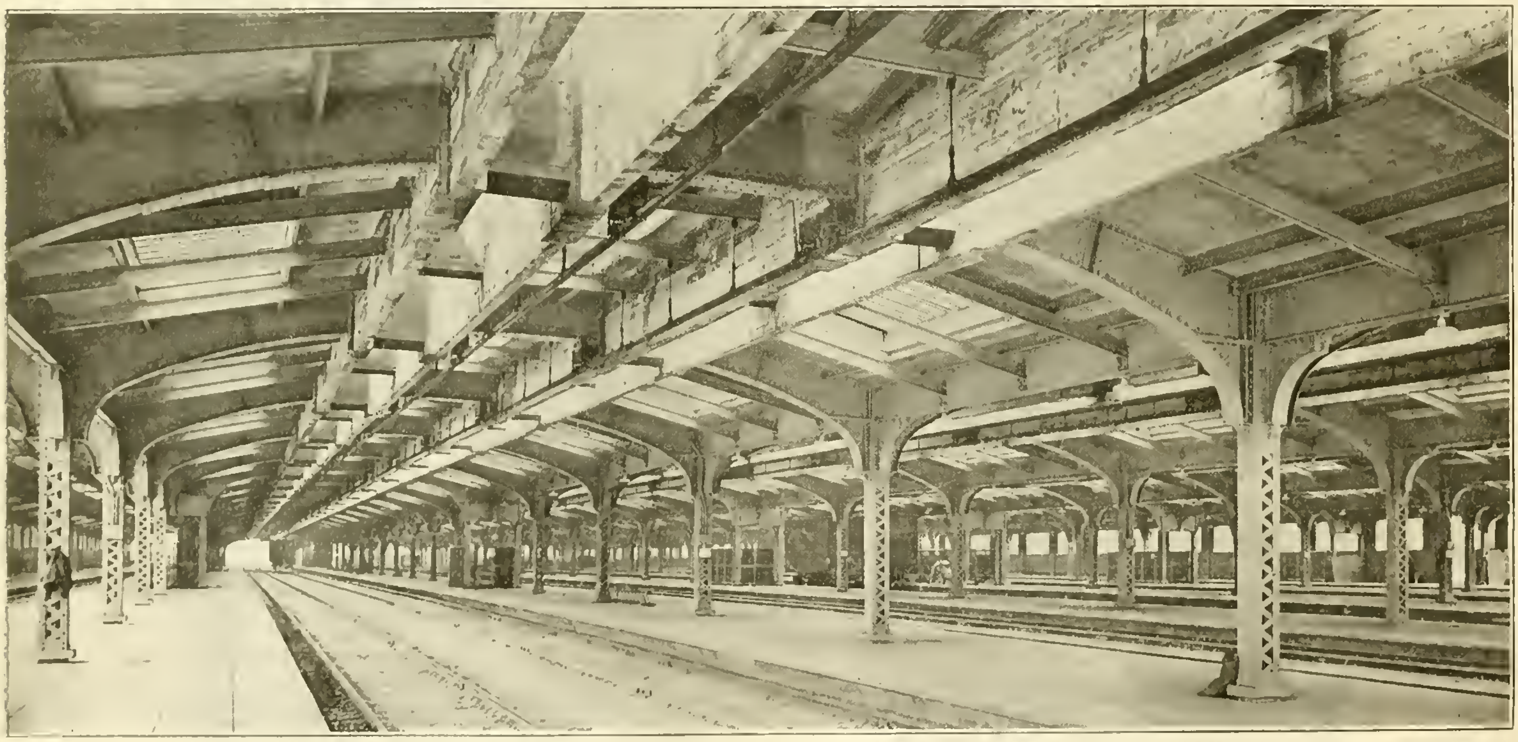


Fig. 3. Interior View of a Portion of the Bush Train Shed of M. C. R. R. Station at Detroit, Mich.

GROUND FLOOR FACILITIES.

The entire ground floor of the building is used for station purposes. There are three entrances, the one on the north side will accommodate those who come in private cars; the one at the west end of the concourse is for those arriving in taxicabs; and the one at the east end of the building will be used by people arriving in street cars. It is estimated that fully 75 per cent of the public will use this entrance, which directs them down the arcade, past the ticket lobby to the ticket windows.

The waiting room (see Fig. 2) is 104 ft. wide by 243 ft. long, with a height of about 63 ft. The floor is of marble tile, and the woodwork, as in all other public spaces, is of East India mahogany. The wainscoting, pilasters and columns are of kasota stone, the walls are of caen stone, and the ceiling consists of Guastavino arches, the color of which is just light enough to blend pleasantly with the marble and ornamental plaster. On the north side of the waiting room, above the main entrance and symmetrical with it, are three large double metal sash windows 21 ft. wide by 40 ft. high. Between the large windows are smaller windows 8 ft. wide by 20 ft. high.

The women's room, which is to the east of the waiting room, is about 52 ft. square. It has a hard-wood parquet floor set in mastic, paneled oak wainscoting, beamed ceiling, and caen stone walls. The woodwork in this room, as in the reading and tea rooms, is of quarter-sawed, flaked white-oak. In connection with this room are the free and pay toilets and dressing rooms. The dressing rooms are equipped with lavatory, tub and shower baths, etc., and are intended to be used by those who wish to change to evening attire for evening appointments without necessitating the use of a hotel room for this purpose.

The reading room, which is to the west of the waiting room, is 39 ft. by 52 ft. This room has a parquet floor, large wood paneled wainscoting, and an ornamental plastered ceiling. The furniture of this room, as well as that of the women's room, is of a rich design to harmonize with the woodwork and is upholstered in green Spanish leather. The atmosphere of this room is one of quiet restfulness, and it is thought that the men who do not wish to smoke and who wish to be relieved of the confusion of the general waiting room will find this pleasant innovation. Writing and reading tables and the best of literature are to be found here.

The tea room is directly to the south of the reading room, and is finished in a manner similar to it. It is possible that this room will be used as a buffet.

The restaurant and lunch room are located to the west of the ticket office and south of the buffet. The lunch room is 39 ft. by 54 ft., and is separated from the restaurant by a quarter-sawed, flaked white-oak paneled screen. It may be entered either from the waiting room or from the concourse. The restaurant is 54 ft. wide by 78 ft. long, with a 9 x 9-in. Welsh quarry tile floor, kasota stone wainscot, caen stone walls, and a domed ceiling of the same material.

The ticket office has a total frontage of about 130 ft., and has 19 ticket windows which are accessible either from the concourse, the ticket lobby, or the waiting room. This office is finished with kasota stone wainscoting, Tennessee marble deal plates, bronze window grilles, and a continuous bronze suitcase rack under the windows.

Above the low roof of the ticket office there is a large electric clock operated from the master clock in the station master's office. This clock is in direct view down the arcade as the passengers enter the east elevator hall. On each side of the arcade there is a bank of three elevators, a total of six cars, five for passenger and one for freight service. The three passenger elevators in the north bank are not being installed at this time. Just west of the north bank of elevators and back of the ticket office are the two main fireproof stair halls extending the entire height of the building, with hose reels on each floor.

To the right as one comes down the arcade to the ticket office is the drug store, telegraph and telephone rooms, and the newspaper booth while to the left is the cigar store, parcel room and information booth.

The barber shop is directly south of the elevator hall. It has a white tile floor, statuary marble wainscoting and trim, and plastered beamed ceiling and walls painted white. Just across the hall are the men's bath and dressing rooms.

The concourse is similar to the waiting room, and has a terrazo floor with marble tile panels. The walls are of light colored face bricks, the same as are found on the exterior of the office building, and are laid in Flemish bond, with panels outlined with light terra cotta strips. The pilaster caps are of the same material.

The molded members of the cornice, beams, and ceiling are of copper, with a large area of ceiling lights which serve to diffuse the light from the skylights above.

There is an ornamental iron fence about 8 ft. high and 110 ft. long, which extends 16 ft. beyond the south wall of the concourse and runs the full width of the ramp. The front of this fence has eight ticket gates and eight train indicators of the National Indicator type. The ends of the fence are equipped with wide folding gates which serve as exits for the people coming from inbound trains. The large majority of the incoming passengers use the east exit and go directly to the trolley cars from the concourse; the remainder take the west exit to the cab office at the west end of the concourse. In this way no confusion between incoming and outgoing passengers is anticipated.

The smoking room is entered from the southeast corner of the concourse. It is about 26 ft. wide by 60 ft. long. The woodwork and the continuous outside seat are of East India mahogany and the floor is of Welsh quarry tile, with plastered walls and ceiling and kasota stone wainscot. Just east of the smoking room is the anteroom and the men's free and pay toilets.

To the west of the ramp is the baggage room, which is accessible from the concourse; while to the east are the offices for the station master and for customs. Below these offices in the basement are the detention rooms.

OFFICE BUILDING.

The high building, which rises above the center of the station, is 345 ft. long and 240 ft. high, the central portion being 54 ft. wide and the wings at each end 108 ft. wide (see Figs. 4 and 5).

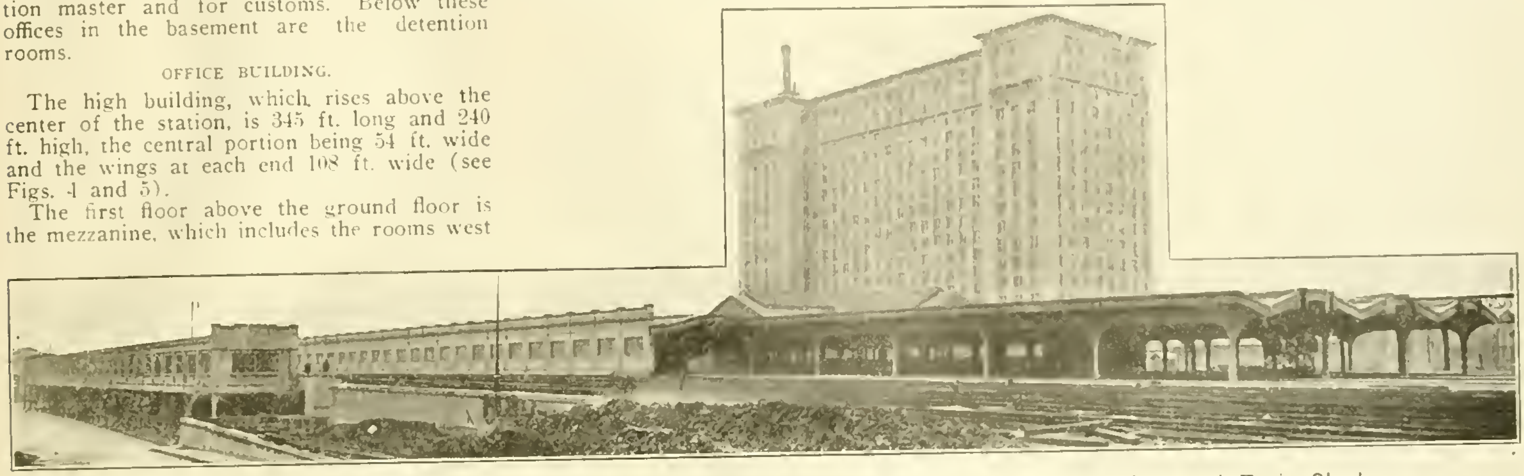


Fig. 4. Rear Elevation cf M. C. R. R. Terminal at Detroit, Mich., Showing Office Building and Train Shed.

The first floor above the ground floor is the mezzanine, which includes the rooms west of the concourse and the paymaster's, telegraph and telephone rooms, etc., to the east of the concourse.

The next floor above this is a storage floor, with a few offices at the east end. At the west end of this floor and directly over the restaurant there is an employees' lunch room which is operated on the self-serve plan.

The next floor is the pipe loft, which will be used for the distribution of the heating and plumbing systems, etc., as well as for a store room.

Above this are 13 office floors, each having an area of about 24,000 sq. ft. The story height for each is 12 ft. 6 ins., and the arrangement of offices is similar. A 12-ft. corridor runs along the central axis of each of these stories. It has a terrazo floor, cove and base, and a high marble wainscoting. All vaults throughout the office building have cement floors. The office floors are of wood laid on a cinder fill supported by a 4-in. reinforced concrete slab. All the woodwork in the offices is of oak, and all partitions are made of plastered terra cotta tiles. The ceilings and walls are painted, and the ribbed glass panels in the upper half of the corridor doors, together with an almost continuous line of transoms, afford good light for the corridors. Small wooden doors are provided to the pipe spaces in the pilasters. The wood base is provided with a wire mold. This mold, together with the picture mold provides a place for all the wires required in the offices. At the east end the corridor widens into an elevator hall in front of the two banks of electric elevators, and back of these elevators there is the main stair hall running the entire height of the building. At the west end there are two more elevators and to the east of them are the typical toilet rooms and another stair hall. Only eight office floors are being finished at this time, but it is expected that the future expansion of the Michigan Central will, in the near future, require the use of the remaining floors.

MECHANICAL PLANT.

The mechanical plant for all the buildings and yards is controlled from the basement of the main building where the boiler plant and the machinery room are located. The boiler room is directly under the cab entrance at the southwest corner of the high building and is equipped with five 450 HP. "Toledo-Flanders" water tube boilers, with provisions for the future installation of another boiler of the same capacity.

The coal for the plant is delivered into a reinforced concrete hopper located at the south end of the service building. The top of the hopper is at the elevation of the rail, and the coal cars switch over the hopper and dump by gravity. The capacity of the hopper is about ten cars. From the bottom of the hopper the coal passes through the coal crusher onto a motor-driven belt conveyor, which delivers it through the large concrete coal and ash tunnel to the electric-driven bucket conveyor. These conveyors work in unison. The bucket conveyor distributes the coal over the continuous coal bunker in front of the boilers, where it is fed to the stoker grates by means of hand chain movable chutes.

The ashes are loaded into the ash cars directly under the boilers. These cars are operated by hand through the tunnel on a narrow-gage track, and are dumped into the ash pit just south of the coal crusher, where the ashes are loaded and are conveyed to the cars on the yard tracks above.

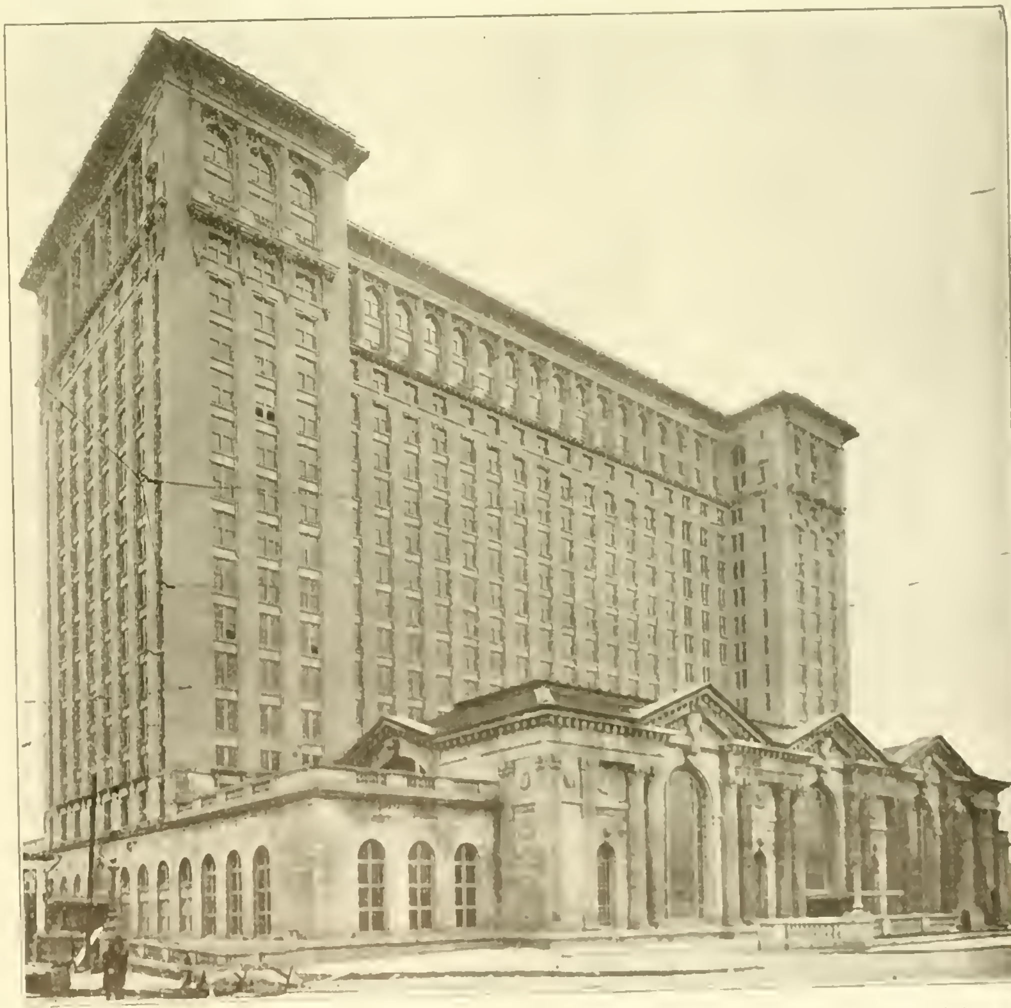


Fig. 5. Front Elevation of M. C. R. R. Station and Office Building at Detroit, Mich.

The machinery room is equipped with the most efficient type of pumps, compressors, incinerator plant, refrigerating system, etc.

The large house and fire tanks are located in the west penthouses of the office building. The surge tank, elevator tank, pneumatic tube tanks, etc., are located in the machinery room. The heating system for the entire office building, from the pipe loft up, is of the down-feed type; while the ground and mezzanine floors are supplied by the up-feed system. Each office floor has 72 radiators, the total radiation for the whole building being about 125,000 sq. ft. The vacuum return is used for the station as well as all the outlying buildings and yards. The station proper is operated under the "Johnson" system of thermostatic control, while the train sheds and other buildings are under hand control.

The building is equipped with a complete fire alarm and protection system, a cold water drinking system, a pneumatic tube system, and a dictaphone system

The piping and wiring have been completed for a watchman's clock system, a vacuum cleaning system, a train announcing system and telautograph system; but these systems will not be installed at this time.

The electric power is 4,600-volt, three-phase alternate current. In order to eliminate as nearly as possible the chances of failure of supply, three independent main feed cables were installed. Two of these come from the Commonwealth-Edison generating plants, and are brought in from opposite sides of the building. The third line is run from the company's sub-station located near the foot of 10th St. and the river. The current first passes through hand controlled switches, down through the transformers and then to the main switchboard or motor generators. Direct lighting is used throughout the terminal, with the exception of the barber shop, where indirect fixtures have been installed.

In wiring the offices provision was made for easy maintenance by use of a removable base mold, back of which the wires are placed. On the floors occupied by the auditor's departments dictagraph plugs have been installed under the desks, and numerous plugs have been installed throughout the building for fans, drop lights, etc.

An automatic telephone system has been installed throughout the various offices.

The electric clock system is controlled by a master clock in the station master's office. At present there are only two clocks installed, in addition to the master clock, the one above the ticket office and the one on the cab stand.

BAGGAGE AND EXPRESS ROOMS.

The baggage rooms occupy about 35,000 sq. ft. in the northwest corner of the terminal. This space is fenced in by means of a wire partition with numerous large gates and is equipped with numerous racks. It is easily accessible from the wagon court on the north and from the private drive on the west. All outside walls are equipped with large Kinnear rolling doors and metal sash windows, giving a large area of openings and good light. The baggage is handled from the various elevators by means of electric trucks. Provision is made so that these loaded trucks can run up the baggage ramp into the baggage checking room in the terminal.

As a continuation of the main ramp in the terminal there is a 40-ft. subway, about 210 ft. long, running under the center of the train shed between the baggage and mail rooms and perpendicular to the tracks. This ramp has a terrazo floor and white terra cotta walls and ceiling construction. Wide, easy stairways lead up from the subway to the tracks on the east and west side.

At the northeast corner of the terminal the mail department and Union News Co. occupy about 17,000 sq. ft. of space adjacent to a wagon court on the north. The American Express Co. has about 30,000 sq. ft. of space on the south side under the freight tracks. All of these areas are equipped in an up-to-date manner and give excellent locations for the purpose for which they are used. The entire floor of the basement is of concrete with a 2-in. mastic covering, which makes an easy and noiseless floor.

There are 19 large hydraulic elevators, each 6 ft. 4 ins. x 15 ft. 4 ins., for the use of all tenants below. These elevators are at both ends of the basement, two on each platform and seven on the express platform. The arrangement of these elevators is such as to do away with almost all of the trucking on the platforms above.

TRAIN SHED.

The train shed covers ten passenger tracks, there being two tracks between each platform and one express track near the south wall. The north and south sides are enclosed with stone and face bricks, laid in a similar manner and of the same color as those used in the construction of the terminal.

The foundation for the train shed substructure consists of a continuous reinforced concrete footing of a 1:2:4 mixture. This, together with the concrete floor slabs supporting the tracks and the beam and girder fireproofing, were poured under the direction of the bridge engineer during the summer of 1910. The concrete used for practically all of the terminal and outlying buildings was also a 1:2:4 mixture.

In designing the train shed and rooms beneath it the amount of headroom was limited by the city sewer below and by the vertical curves of the tracks above, as they emerge from the tunnels. Consequently the clear headroom of the rooms below is only about 10 ft.

The portion of the train shed above grade is of the "Bush" type, and is 246 ft. wide by 1,106 ft. long. Figure 3 shows an inside view of a portion of the train shed and indicates the type of construction used. The superstructure rests on latticed steel columns spaced 40 ft. on centers north and south and 35 ft. east and west. The trusses are curved, and support a 2½-in. concrete roof slab on "Hy-rib," with ½ in. of plaster underneath.

About 28 per cent of the entire roof is of the "Anti-Pluvius" skylight, set higher on the roof than is usual to overcome the trouble caused by water backing up in the gutters. An adequate system of drainage has been installed and every precaution taken to overcome the leaks over the expansion joints. Steam jets have been installed in the drain stacks to overcome the trouble of freezing in cold weather.

Two cross-overs on each track have been installed with "Transit" smoke jacks above to provide for the breaking up of trains in short order. These smoke jacks will not be used, however, unless steam locomotives are used for the switching.

The low platform type has been used and the clearance from top of rail to bottom of smoke duct is 15 ft. 6 ins. The platforms are 19 ft. wide, and extend for some distance beyond the ends of the shed. The platforms are of concrete, with a liberal supply of vault lights, their maximum length being about 1,400 ft. The field of the platforms is of a dull red color and the borders are dark gray, all marked at about 2 ft. centers both ways. Carborundum was used in the finish to obtain an effective wearing surface. The whole yard is equipped for electric locomotives to conform with the equipment of the tunnels. All trains are hauled through the tunnels by means of the electric locomotives. Figure 4 is a rear view of the terminal and shows the completed train sheds.

EXPRESS OFFICE, WARMING HOUSE AND SERVICE BUILDING.

The express office is south of the seven freight tracks which are not under cover, and is at the same elevation as these tracks. This building is one story high, and is 21 ft. wide by 530 ft. long. It is finished with metal sash windows, wood floors, terra cotta tile walls, plastered on both sides, and a wood finish to match that of the terminal. The offices are equipped with numerous wire screen partitions, booths, wood counters, toilets, lavatories, etc., which make them the most desirable the company has ever had.

The warming house is a one-story building about 40 ft. wide by 320 ft. long, and is located at 20th St. and the tracks. About 150 ft. of it is parallel to 20th St., and is used as a wood shop, oil and repair shed, etc. The wing parallel to the track is 170 ft. long, and is used for warming Pullman cars. There is an inspection pit under each track,, and the building has enough radiation to keep the temperature at 100° F. during zero weather. It is possible to thaw out four coaches in a very few minutes.

Just north of this structure is the wheel pit building. This is used for the removal of flat wheels, etc., and it is equipped so as to allow a pair of trucks to be changed in 15 minutes.

The service building is just west of the main building, across 17th St. It is 35 ft. wide by 260 ft. long and is three stories high. This building, as well as the warming building, is finished outside to harmonize with the terminal. It houses the Pullman stores, the cleaners' and porters', and the linen and commissary departments, and the various smaller offices and storerooms of the Michigan Central.

Just west of the service building is the coach yard, which is equipped with tunnels for steam, water and air connections, etc., as well as various truck platforms between the tracks. The capacity of the yard is about 140 coaches. All brake testing is done in this yard, and all of the water, steam, air, etc., comes from the terminal through the coal and ash tunnel. This yard with its buildings is considered one of the best equipped and up-to-date yards in the country.

PERSONNEL.

The whole project was carried out under the direction of Mr. H. B. Ledyard, vice-president, Detroit River Tunnel Co. The terminal building was designed by Reed & Stem and Warren & Wetmore, architects, under the direction of Mr. George H. Webb, chief engineer, Michigan Central R. R. and the Detroit River Tunnel Co. The steelwork for the terminal building was designed by Balcom & Darrow, engineers. The train shed and the outlying buildings were designed and the construction work directed by the railroad's architect, Mr. Edward W. Smith. Mr. W. B. Goddard, Jr., was engineer in charge of the inspection, being assisted by the writer.

The George A. Fuller Co., of New York, was the general contractor. The steelwork was fabricated by the McClintic Marshall Construction Co., of Pittsburgh.

A Summary of the Results of a Series of Tests to Determine the Bond Between Concrete and Steel.

An elaborate series of tests has been made at the Engineering Experiment Station, University of Illinois, to secure additional information on the nature of the bond resistance of reinforcing bars in concrete, to determine values of bond resistance for a wide range of conditions, and to study bond action in specimens of different forms. Tests were made on pull-out specimens and on reinforced concrete beams. In both forms of specimens attention was given to obtaining accurate measurement of the slip of bar through the concrete as the loading progressed. In many of the beam tests the slip of bar at various points along its length was measured for different loads. In a few of the beam tests the distribution of bond stress was studied by measuring the changes in stress in the longitudinal reinforcement throughout the length.

In the pull-out tests the concrete block was generally 8 ins. in diameter and 8 ins. long, with the rod buried axially. The size of bar varied from ¼-in. to 1¼-in. These pull-out tests covered a wide range and included the effect of dimensions of specimen, of form of bar, of conditions of storage, of age and mix (using both plain and deformed bars), and of different methods of loading. They also considered the bond resistance of concrete setting under pressure, the effect of reapplied loads, a comparison with bond resistance of reinforced concrete beams, etc. The tests with deformed bars were intended to bring out the action of the deformed bar as contrasted with the plain bar and not to determine the value of particular forms of bars.

A special effort was made to determine the behavior of beams subjected to high bond stresses. The beams tested were 8x12 ins. in section, with an effective depth of 10 ins., the span length generally being 6 ft., although a few beams were tested with span lengths of 5 to 10 ft. All beams were tested with two symmetrical loads, generally at the one-third points of the span. With the exception of six tests, the longitudinal reinforcement consisted of a single bar of large diameter placed horizontally throughout the length of the beam, both plain and deformed bars being used.

A description of the tests and of the results obtained has been published as Bulletin No. 71, "Tests of Bond Between Concrete and Steel," by Duff A. Abrams, the work being done under the direction of Professor A. N. Talbot. As the tests covered a wide range of conditions, the results have a significant bearing on the nature of bond resistance, the action of bars of different forms under bond stress, and the behavior of beams subjected to high bond stresses. The load-slip determinations have given definite information on the nature and distribution of bond resistance. The following is a summary of the principal observations and conclusions of this series of tests. Most of the conclusions are based on comparisons involving the load-slip relations, although in a few of the tests the bond stress was determined from a study of the variations in the tensile stress in the reinforcing bar. The latter method furnishes a much more direct means of measuring the bond stress, but it has been available only since the recent development of a non-fixed extensometer.

SUMMARY OF RESULTS OF PULL-OUT TESTS.

(1) Bond between concrete and steel may be divided into two principal elements, adhesive resistance and sliding resistance. The source of adhesive resistance is not known, but its presence is a matter of universal experience with materials of the nature of mortar and concrete. Sliding resistance arises from inequalities of the surface of the bar and irregularities of its section and alignment, together with the corresponding conformations in the concrete. The adhesive resistance must be overcome before sliding resistance comes into action. In other words, the two elements of bond resistance are not effective at the same time at a given point. Many evidences of the tests indicate that adhesive resistance is much the more important element of bond resistance.

(2) Pull-out tests with plain bars show that a considerable bond stress is developed before a measurable slip is produced. Slip of bar begins as soon as the adhesive resistance is overcome. After the adhesive resistance is overcome, a further slip without an opportunity of rest is accompanied by a rapidly increasing bond stress until a maximum bond resistance is reached at a definite amount of slip.

(3) The true relation of slip of bar to bond stress can best be studied by considering the action of a bar over a very short section of the embedded length. The difficulties arising from secondary stresses made it impracticable to conduct tests on bars embedded for very short lengths. The desired results were obtained by varying the forms of the specimens in such a way that the effect of different combinations of dimensions could be studied.

(4) Pull-out tests with plain bars of the same size embedded different lengths furnish data which suggest the values of bond resistance over a very short length of embedment, or indicate values of bond resistance which are independent of the length of embedment. Tests with bars of different size which were embedded a distance proportional to their diameters give the true relation when the effect of size of bar is eliminated. Two series of tests of this kind on plain round bars of ordinary mill surface gave almost identical values for bond resistance after eliminating the effect of length of embedment and size of bar, and we may consider that these values represent the stresses which were developed in turn over each unit of area of the embedded bar as it was withdrawn by a load applied by the method used in these tests. These tests showed that for concrete of the kind used (a 1-2-4 mix, stored in damp sand and tested at the age of about 60 days) the first measurable slip of bar came at a bond stress of about 260 lbs. per square inch, and that the maximum bond resistance reached an average value of 440 lbs. per square inch. If we conclude that adhesive resistance was overcome at the first measurable slip, it will be seen that the adhesive resistance was about 60 per cent of the maximum bond resistance. This ratio did not vary much for a wide range of mixes, ages, size of bar, condition of storage, etc.

(5) Sliding resistance reached its maximum value for plain bars of ordinary mill surface at a slip of about 0.01 in. The constancy in the amount of slip corresponding to the maximum bond resistance for a wide range of mixes, ages, sizes of bar, conditions of storages, etc., is a noteworthy feature of the tests. With further slip the sliding resistance decreased slowly at first, then more rapidly, until with a slip of 0.1 in. the bond resistance was about one-half its maximum value.

(6) Pull-out tests with plain round bars show end slip to begin at an average bond stress equal to about one-sixth the compressive strength of 6-in. cubes from the same concrete; the maximum bond resistance is equal to about one-fourth the compressive strength of 6-in. cubes. These values were about the same for a wide range of mixes, ages and conditions of storage. In terms of the compressive strength of 8x16-in. concrete cylinders these values would be about 13 per cent for first end slip and 19 per cent for the maximum bond resistance.

(7) The tests indicate that bond stress is not uniformly distributed along a bar embedded any considerable length and having the load applied at one end. Slip of bar begins first at the point where the bar enters the concrete, and the bond stress must be greater here than elsewhere until a sufficient slip has occurred to develop the maximum bond resistance at this point. Slip of bar begins last at the free end of the bar. After slip becomes general, there is an approximate equality of bond stress throughout the embedded length.

(8) Small bars gave a bond resistance somewhat higher than the large bars during the early stages of the test. This was probably on account of greater irregularity of section and alignment of the smaller bars. The maximum bond resistance was not materially different for bars of different diameters.

(9) Computations based on the elastic properties of the materials indicate that in the pull-out tests the tensile deformation in the bar had a much greater effect on the amount of bond stress which permitted a given slip of bar than had the compressive deformation in the concrete block in which the bar was embedded.

(10) Rusted bars gave bond resistances about 15 per cent higher than similar bars with ordinary mill surface.

(11) The tests with flat bars showed wide variations of bond resistance and were not conclusive. Square bars gave values of unit-stress about 75 per cent of those obtained with plain round bars.

(12) T-bars gave lower unit bond resistance than plain round bars, but gave about double the bond resistance per unit of length that was found for the plain round bars of the same sectional area.

(13) With polished bars the bond resistance is due almost entirely to adhesion between the concrete and steel. Numerous tests with polished bars embedded in 1-2-4 concrete and tested at 60 days indicated a maximum bond resistance of about 160 lbs. per square inch, or about 60 per cent of the bond resistance of bars of ordinary surface at small amounts of slip. This value agrees closely with tests reported elsewhere, and apparently represents the value of the tangential adhesion between any clean steel and concrete of this quality. The sliding resistance of polished bars was very low.

(14) Tests with polished bars with wedging and non-wedging tapers showed that adhesion was broken for both types of bar at about the same bond stress as in the polished bars of uniform section.

(15) The tests with polished bars with wedging taper showed that after the adhesion was broken a considerable movement of the bar (as much as ¼-in. with the smallest tapers) was required before the bond resistance again reached the amount which was at first carried by the adhesive resistance. The amount of movement necessary to restore the bond stress to the value of the original adhesive resistance was inversely proportional to the amount of taper. This indicates that a definite normal compression must be developed in the surrounding concrete before a longitudinal component equivalent to the original tangential adhesion is produced.

(16) It was noted in the tests with plain bars that sliding resistance was due to inequalities of the surface of the bar and to irregularities of its section and alignment. The projections on a deformed bar give an exaggerated condition of inequality of surface or irregularity of section. Adhesive resistance must be destroyed and the usual sliding resistance largely overcome and the concrete ahead of the projections must undergo an appreciable compressive deformation before the projections on a deformed bar become effective in taking bond stress. The tests indicate that the projections do not materially assist in resisting a force tending to withdraw the bar until a slip has occurred approximating that corresponding to the maximum sliding resistance of plain bars. As slip continues a larger and larger portion of the bond stress is taken by direct bearing of the projections on the concrete ahead.

(17) In determining the comparative merits of deformed bars, the bar which longest resists beginning of slip should be rated highest, other considerations being equal. The bond stresses developed at an end slip of 0.001 in. furnished the principal basis of comparison for the different types of deformed bars. At an end slip of 0.001 in. 12 sets of deformed bars of ¾-in. and larger sizes embedded 8 ins. in 1-2-4 concrete, tested at about two months, developed an average bond resistance of 318 lbs. per square inch, 4 per cent higher than the corresponding value for plain bars. At this stage of the test, two sets of deformed bars gave practically the same bond resistance, five sets gave lower values, and five sets higher values than the plain rounds. At an end slip of 0.01 in., corresponding to the maximum bond resistance of plain bars, the average bond resistance of the 12 sets of deformed bars was 445 lbs. per square inch, 10 per cent higher than plain rounds. At this stage of the test two sets gave about the same values, two sets gave lower values, and eight sets gave higher values than the plain bars. The hooping used in these specimens had a marked effect in increasing the bond resistance even at small amounts of slip.

(18) The concrete cylinders of the pull-out specimens with deformed bars were reinforced against bursting or splitting, because it was desired to study the load-slip relation through a wide range of values. The bond stresses corresponding to an end slip of 0.1 in. are the highest stresses reported for the deformed bars. In only a few tests was the maximum bond resistance reached at an end slip less than 0.1 in. It should be recognized that, in general, the bond stresses reported for deformed bars at end slip of 0.05 and 0.1 in., could not have been developed with bars embedded in unreinforced blocks. These high values of bond resistance must not be considered as available under the usual conditions of bond action in reinforced concrete members. In the tests in which the blocks were not reinforced, evidence of splitting of the blocks was found at end slips of 0.02 to 0.05 in.

(19) The normal components of the bearing stresses developed by the projections on deformed bar may produce very destructiv bursting stresses in the surrounding concrete The bearing stress between the projection and the concrete in the tests with certain type of commercial deformed bars was computed t be from 5,800 to 11,000 lbs. per square inc at the highest bond stresses considered i these tests. For bars having projections different heights and spacing, the bearin stresses on the projections at the highest bon stresses considered were inversely proportiona to the bond stress which had been develope by the bar at an end slip of 0.01 in., the sli at which the projections were beginning to b effective. These considerations show that th ratio of the area of the projections measure at right angles to the bar to the superficia area of the bar in the same length is th proper criterion for judging of the effectiv bond resistance of a deformed bar. In som forms of bar the bearing stresses must hav been much higher than the values given abov

The large slip and the high bearing stresses developed in the later stages of the tests show the absurdity of seriously considering the extremely high values that are usually reported to be the true bond resistance of many types of deformed bars.

(20) Round bars with standard V-shaped threads gave much higher bond resistance at low slips than the commercial deformed bars. The average bond resistance at an end slip of 0.001 in. was 612 lbs. per square inch. The maximum bond resistance was 745 lbs. per square inch. These were the only deformed bar tests in which failure came by shearing the surrounding concrete.

(21) In a deformed bar of good design the projections should present bearing faces as nearly as possible at right angles to the axis of the bar. The areas of the projections should be such as to preserve the proper ratio between the bearing stress against the concrete ahead of the projections and the shearing stress over the surrounding envelope of concrete. Failure by shearing of the concrete should be avoided. The tests indicate that the areas of the projections measured at right angles to the axis of the bar should not be less than, say, 20 per cent of the superficial area of the bar. A closer spacing of the projections than is used in commercial deformed bars would be of advantage. Advocates of the deformed bar would do well to recognize the fact that in a deformed bar which may be expected to develop a high bond resistance, a certain amount of metal must be used in the projections which probably will not be available for taking tensile stress.

(22) The 1-in. twisted square bars gave a bond resistance per unit of surface at an end slip of 0.001 in., only 88 per cent of that for the plain rounds. Following an end slip of about 0.01 in., these bars showed a decided decrease in bond resistance, and a slip of five to ten times this amount was required to cause the bond resistance to regain its first maximum value. After this, the bond resistance gradually rose as the bar was withdrawn. Some of the bars were withdrawn 2 or 3 ins. before the highest resistance was reached. The apparent bond stresses at these slips were very high; but, of course, such stresses and slips could not be developed in a structure and could not have been developed in the tests had the blocks not been reinforced against bursting. Such values are entirely meaningless under any rational interpretation of the tests.

(23) The load-slip curves for twisted square bars are similar to those for polished bars with wedging taper. The twisted bar is essentially a combination of the wedging and non-wedging taper. As the bar is drawn through the concrete the wedging tapers are drawn more firmly against the concrete ahead, while at the same time the non-wedging tapers are separated from the concrete with which they were originally in contact. The drop in the load-slip curves after an end slip of about 0.01 in. shows that the separation of about one-half of the surface of the bar from its original contact and the continued sliding of the flatter portions of the bar, until a large slip has occurred, have a greater influence in reducing the average bond resistance than the increased bearing of the wedging tapers has in raising the bond resistance. The results found with the twisted square bar do not justify its present wide-spread popularity as a reinforcing material.

(24) The tests with plain round bars anchored by means of nuts and washers and with washers only showed that the entire bar must slip an appreciable amount before these forms of anchorage come into action. Anchorages of the dimensions used in these tests did not become effective until the bar had slipped an amount corresponding to the maximum bond resistance of plain bars. With further movement the apparent bond resistance was high, but was accompanied by excessive bearing stresses on the concrete.

(25) The load-slip relation for bars anchored by means of hooks and bends was not determined. The high resistance given in these tests was probably a result of the bearing stresses developed in the concrete ahead of the bends.

(26) Tests on specimens stored under different conditions indicate that concrete stored in damp sand may be expected to give about the same bond resistance and compressive resistance as that stored in water. Water-stored specimens gave values of maximum bond resistance higher in each instance than the air-stored specimens; the increase for water storage ranged from 10 to 45 per cent. The difference seemed to increase with age. The presence of water not only did not injure the bond for ages up to three years, but it was an important factor in producing conditions which resulted in high bond resistances. However, it was found that specimens tested with the concrete in a saturated condition gave lower values for bond than those which had been allowed to dry out before testing. The bars in specimens which had been immersed in water as long as 3½ years showed no signs of rust or other deterioration.

(27) Specimens made out-doors in freezing weather, where they probably froze and thawed several times during the period of setting and hardening, were almost devoid of bond strength.

(28) Pull-out tests made at early ages gave surprisingly high values of bond resistance. Plain bars embedded in 1-2-4 concrete and tested at 2 days did not show end slip of bar until a bond stress of 75 lbs. per square inch was developed. Bond resistance increases most rapidly with age during the first month. The richer mixes show a more rapid increase than the leaner ones. The tests on concrete at ages of over one year showed that the bond resistance of specimens stored in a damp place may be expected ultimately to reach a value as much as twice that developed at 60 days.

(29) The load-slip relation of leaner and richer mixes was similar to that for 1-2-4 concrete. For a wide range of mixes the bond resistance was nearly proportional to the amount of cement used. This relation did not obtain in a mix from which the coarse aggregate had been omitted.

(30) When the application of load was continued over a considerable period of time or when the load was released and reapplied, the usual relation of slip of bar to bond resistance was considerably modified. The few tests which were made indicate that the bond stress corresponding to beginning of slip is the highest stress which can be maintained permanently or be reapplied indefinitely without failure of bond. The effect of continued and repeated load, impact, etc., may well be the subject of further experimental study.

(31) Little difference was found in the pull-out tests whether the load was distributed over the entire face of the blocks or over a narrow ring at the center of the block or around the edge of the face of the block.

(32) Specimens molded in a horizontal position gave lower bond resistance than those molded in a vertical position; when settlement of the bar with the settlement of the concrete was entirely prevented, the bond resistance was reduced to about 60 per cent of that found for similar specimens which were molded with the bars in a vertical position. Plain bars tested by being pulled in the same or the opposite direction from the settlement of the concrete during setting gave about the same bond resistance, but in the tests of certain deformed bars this was not true.

(33) The term "autogenous healing" is used to designate phenomena observed in pull-out tests and in compression tests of concrete cylinders in which the hardening of the concrete was interrupted by loading the specimen at early ages to its ultimate resistance. Up to an age of one year the bond resistance of specimens stored in damp sand was not affected by as many as four loadings at intervals during the period of storage up to the ultimate resistance. For specimens stored in air and tested in the same way, the bond resistance was less than for damp sand storage, but the tests showed a steady increase in bond resistance with each loading up to three months. Specimens which had been stored in air for two months before the first test and in water thereafter showed a decrease in bond with each subsequent loading, although the bond resistance in the last test was fairly high. The presence of water apparently permits the continuation of the hydraulic action of the cement for several months after the mixing of the concrete.

EFFECT OF APPLYING PRESSURE ON CONCRETE DURING SETTING.

(34) Bond resistance of plain bars is greatly increased if the concrete is caused to set under pressure. With a pressure of 100 lbs. per square inch on the fresh concrete for five days after molding, the maximum bond resistance was increased 92 per cent over that of similar bars in concrete which had set without pressure. The greater density of the concrete and its more intimate contact with the bar seems to be responsible for the increased bond resistance. Light pressures gave an appreciable increase in bond resistance. With polished bars the effect of pressure was slight.

(35) As might have been expected, the compressive resistance of concrete setting under pressure was increased in much the same ratio as the bond resistance. At the age of 80 days the initial modulus of elasticity in compression for concrete which set under a pressure of 100 lbs. per square inch was about 37 per cent higher and the compressive strength was increased by about 73 per cent over that of concrete which had set without pressure. The density of the concrete, as determined by the unit weights, was increased about 4 per cent by a pressure of 100 lbs. per square inch on the fresh concrete. The increase in strength and density was relatively greater for the low than for the high pressures. A pressure continued for one day, or until the concrete had taken its final set and hardening had begun, seems to have produced the same effect in increasing the strength and elastic properties of the concrete as when the pressure was continued for a much longer period.

(36) Concrete cylinders tested in compression at age of 80 days after having been loaded to failure at 7 days gave compressive strengths nearly as high as those tested for the first time at the same age. Retests of cylinders which had set under pressure gave similar results.

SUMMARY OF RESULTS OF BEAM TESTS.

(37) Beams of comparatively short span, reinforced with bars of large size, were used in order to develop high bond stresses and give bond failures. Most of the beams failed in bond; a few failed by a combination of bond and diagonal tension or by tension in the steel.

(38) The usual method of computing the bond stress in a reinforced concrete beam does not take account of all the phenomena of bond action. Slip of bar due to beam bond action and the presence of anti-stretch slip may be expected to greatly modify the distribution of bond stress over the length of the bar, and otherwise to affect resistance to beam bond stresses. However, the nominal values for bond resistance, computed by the usual formula, form a useful basis for comparison in beams in which the dimensions and general make-up are similar.

(39) Slip of bar was a phenomenon in all beam tests in which careful slip observations were made. These load-slip relations give important indications as to the bond stress developed at points along the length of the beam.

(40) Slip was first observed in the middle region of the span at loads producing a tensile stress in the steel of about 6,000 lbs. per square inch. In this region the shear is zero and hence beam bond action, as usually understood, is absent. As the load was increased, slip of bar progressed through the outer thirds toward the ends of the beam at a rate nearly proportional to the increase of load. After slip occurred at the ends, the outer thirds of the length of the bar moved toward the middle of the span relative to the adjacent concrete. Slip of bar was probably partly responsible for the opening of outer cracks,

since slipping was observed in the outer thirds of the beams before the cracks became visible.

(41) The mean computed values for bond stresses in the 6-ft. beams in the series of 1911 and 1912 were as given below. All beams were of 1-2-4 concrete, tested at two to eight months by loads applied at the one-third points of the span. Stresses are given in pounds per square inch.

Type of bar.	Number of tests.	First end slip of bar.	End slip of 0.001 in.	Maximum bond stress.
1 and 1¼-in. plain round..	28	245	340	375
¾-in. plain round..........	3	186	242	274
⅝-in. plain round..........	3	172	235	255
1-in. plain square..........	6	190	248	278
1-in. twisted square........	3	222	289	337
1⅛-in. corrugated round..	9	251	360	488

(42) In the beams reinforced with plain bars end slip begins at 67 per cent of the maximum bond resistance; for the corrugated rounds this ratio is 51 per cent, and for the twisted squares, 66 per cent.

(43) The bond unit resistance in beams reinforced with plain square bars, computed on the superficial area of the bar, was about 75 per cent of that for similar beams reinforced with plain round bars of similar size.

(44) Beams reinforced with twisted square bars gave values at small slips about 85 per cent of those found for plain rounds. At the maximum load, the bond-unit stress with the twisted bars was 90 per cent of that with plain round bars of similar size.

(45) In the beams reinforced with 1⅛-in. corrugated rounds, slip of the end of the bar was observed at about the same bond stress as in the plain bars of comparable size. At an end slip of 0.001 in., the corrugated bars gave a bond resistance about 6 per cent higher and at the maximum load, about 30 per cent higher than the plain rounds.

(46) The beams in which the longitudinal reinforcement consisted of three or four bars smaller than those used in most of the tests gave bond stresses which, according to the usual method of computation, were about 70 per cent of the stresses obtained in the beams reinforced with a single bar of large size. The progressive opening of cracks with increase in load was well shown in these tests. These beams showed cracks nearer the ends than usual. The distances of the outermost cracks from the ends of the beams suggest that the unbroken length of embedment has an important bearing on the maximum loads which the beams may be expected to carry before failing by bond. It seems probable that the lower computed bond stresses in these tests are due to errors in the assumptions made as to the distribution of bond stress and not to actual differences of bond resistance in the bars of different size.

(47) The tests on beams with the loads placed in different positions with respect to the span gave little variation in bond resistance during the early stages of the tests. The maximum bond resistances increased rapidly as the load approached the supports. These tests indicate that the variation in the maximum bond stresses must be due to the presence of other than normal beam action.

(48) Nearly all the beams tested on span lengths of 7 to 10 ft. failed by tension in the steel and did not develop the maximum bond resistance, although high bond stesses were obtained. The bond stress corresponding to first end slip of bar did not vary much with the span length.

(49) The bond stresses developed in the beam tests indicate that with beams of the same cross-section the bond stresses are distributed in the same way during the early stages of the test in beams varying widely in span length and loading. During the later stages of the test, the distribution of bond stress seems to depend largely upon the conditions of stress in the concrete through the region of the span where beam bond stresses are high. The distribution of bond stresses in beams of different cross-section apparently varies with the relative dimensions of the beam and the reinforcing bars.

(50) The use of auxiliary tensile reinforcement in the outer thirds of the beam served to modify the distribution of bond stress during the early stages of the test, but did not have any influence on the maximum bond resistance. While the auxiliary bars seemed to prevent the opening of outer cracks, the tests indicate that interior cracks which did not appear on the surface of the beam may have opened to an extent that permitted the same distribution of bond stress as was found in other tests.

(51) Increasing the thickness of the concrete below the reinforcing bars beyond the depth usually employed caused a very large increase in the resistance to bond and web stresses. The added stiffness of the beam and the increased flexural strength through the outer thirds of the span, prevented the formation of cracks in these regions. In the other beam tests such cracks were found to interrupt the continuity of bond action and to be an important factor in producing lower average bond resistances.

(52) Increasing the length of overhang of the ends of the beam beyond the support did not increase the resistance to web stresses as indicated by the opening of outer cracks, but it had an influence on the bond resistance. The bond resistance at first end slip was greater in the beams with the longer overhang. The maximum bond resistance was materially increased by the additional overhang.

(53) In the reinforced concrete beams it was found that very small amounts of slip at the ends of the bar represented critical conditions of bond stress. For beams failing in bond the load at an end slip of 0.001 in. was 89 per cent to 94 per cent of the maximum load found in beams reinforced with plain bars, and 79 per cent of the maximum load for similar beams reinforced with corrugated bars. As soon as slip of bar became general, other conditions were introduced which soon caused the failure of the beam.

(54) The bond stresses developed in a reinforced concrete beam by a load applied as in these tests varies widely over the region in which beam bond stresses are present. High bond stresses are developed just outside the load points at comparatively low loads. The load which first developed a bond stress nearly equal to the maximum bond resistance in the region of beam bond stresses produced a stress near the support which was not more than about 15 to 40 per cent of the maximum bond resistance. As the load is increased, the region of high bond stress is thrown nearer and nearer the support, and at the same time the bond stress over the region just outside the load point becomes steadily smaller. This indicates a piecemeal development of the maximum bond stress as the load is increased. The actual bond stresses in certain tests varied from less than one-half to more than twice the average bond resistance computed in the usual manner.

(55) Slip of bar in a reinforced concrete beam has a marked influence in increasing the center deflection during the later stages of loading.

(56) The comparison of the bond stresses developed in beams and in pull-out specimens from the same materials is of interest. Such a comparison should be made for similar amounts of slip. In the pull-out tests the maximum bond resistance came at a slip of about 0.01 in. for plain bars. The mean bond resistance for the deformed bars tested was not materially different from that of the plain bars until a slip of about 0.01 in. was developed; with a continuation of slip the projections came into action and with much larger slip high bond stresses were developed. The beam tests showed that about 79 to 91 per cent of the maximum bond resistance was being developed when the bar had slipped 0.001 in. at the free end; hence the bond stress developed at an end slip of 0.001 in. was used as a basis of the principal comparisons in the pull-out tests. However, it is recognized that under certain conditions, the stresses developed at larger amounts of slip may have an important bearing on the effective bond resistance of the bar.

(57) The pull-out tests and beam tests gave nearly identical bond stresses of similar amounts of slip in many groups of tests, but it seems that this was the result of a certain accidental combination of dimensions in the two forms of specimens and did not indicate that the computed stresses in the beams were the correct stresses. However, it is believed that a properly designed pull-out test does give the correct value of bond resistance, and gives values which probably closely represent the bond stresses which actually exist in a beam or other member as slipping is produced from point to point along the bar. The relative position of the bar during molding may be expected to influence the values of bond resistance found in the tests.

(58) A properly made pull-ont test on a specimen of correct design is a valuable aid in determining the bond resistance of reinforcing steel in concrete, if due consideration is given to the load-slip relation. The tensile stress in the bar should be kept well below the elastic limit. Best results will be obtained by using a relatively short embedment. An embedment of 8 diameters is recommended.

(59) A working bond stress equal to 4 per cent of the compressive strength of the concrete tested in the form of 8x16-in. cylinders at the age of 28 days. (equivalent to 80 lbs. per square inch in concrete having a compressive strength of 2,000 lbs. per square inch) is as high a stress as should be used. This stress is equivalent to about one-third that causing first slip of bar and one-fifth of the maximum bond resistance of plain round bars as determined from pull-out tests. The use of deformed bars of proper design may be expected to guard against local deficiencies in bond resistance due to poor workmanship and their presence may properly be considered as an additional safeguard against ultimate failure by bond. However, it does not seem wise to place the working bond stress for deformed bars higher than that used for plain bars.

Iron and Steel Exhibit at the Panama Exposition.—The iron and steel industry will be given a prominent place among the various attractions at the Panama Pacific International Exposition at San Francisco next year by the comprehensive exhibit of the United States Steel Corporation and subsidiary companies.

A general committee headed by A. T. De-Forest of the U. S. Steel Products of San Francisco has been directing the work, and H. V. Jamison, advertising manager of the American Sheet & Tin Plate Company of Pittsburgh, has just been appointed director of exhibits for the entire Corporation.

The process of manufacturing iron and steel products will be shown, from the mines where the ore is taken out of the ground, transportation of ore by rail and water, dock operations, and the production of coal, coke and pig iron, to the specialized lines of manufacture. The exhibit will even go so far as tc show how the by-products are produced anc will show many of the products in practica use

A motion picture show will be part of the exhibit, and through this medium will be portrayed the operations of the Corporatior throughout all its departments. The Corporation's many devices for insuring safety of employes will be exhibited and data will b given as to the enormous amounts expende for this purpose.

Preservation of wooden telegraph poles giving way at the base is being accomplished b the East St. Louis & Suburban Railway b placing con rete collars around the rotte parts. The first method of transporting th material for the concrete to the site, unloading, mixing and placing by hand proved to costly. As a solution, a mixer was placed o a flat car provided at one end with a cab fo housing the control apparatus. The car an mixer are operated by one man and the con rete conveyed to place by means of a lon trough. This method has reduced the cos one-half.

Drainage and Irrigation

Losses of Water in Irrigation Systems.

Contributed by P. M. Fogg, U. S. Reclamation Service, from an Address Before the Conference of Operating Engineers at Boise, Idaho.

The matter of economy in the use and handling of water for irrigation is one which has attained significance in this country only in recent years, and is, in fact, even at the present time usually given too little consideration, except where necessity is the controlling factor. The early history of irrigation in the United States affords an excellent example for the argument of the conservationist, for the pioneers in irrigation farming, with lavish use of apparently inexhaustible water supplies, have gradually impaired or seeped and rendered worthless many thousands of acres of land in the arid west. Fortunately, most of the sources of water for irrigation are perennial; and the run-off may be stored. Then, too, scientific knowledge is concerning itself with the drainage of swamped and alkalied lands, so that in time it may be expected that the prodigality of the past will, in some measure, be atoned for.

The tremendous growth during the past 15 years of irrigation enterprises in this country has resulted in a more thorough realization of the fact that even with the most careful and economical use of the water available for the irrigation of our arid lands there will always remain vast areas which cannot feasibly be artificially watered. It is becoming of increasing importance, therefore, that methods of true economy shall be employed in the management and utilization of water as applied to the irrigation of land.

There are many ways in which economy can be introduced in the use and handling of water, but this discussion is confined principally to the subject of losses from canals and their reduction or prevention. There can be no difference of opinion as to the desirability in general of reducing or eliminating the losses which occur as a feature of operation in nearly all large canal systems. Aside from the value of the water lost, which may or may not be very significant to the beneficiaries of the water right, but which is of undoubted public importance, there is the tendency for the subsurface water plane, affected by percolation from the canals, to raise to a dangerous extent, carrying deposits of alkaline salts to the surface of the soil. Except, therefore, in unusual cases, such as that of the Egin bench in Idaho, where percolation from the distributaries assists in successful sub-irrigation of the lands without apparent detriment, the losses constitute an unfortunate feature of operation.

The relative importance of the losses to the owners of any particular irrigation system is intimately related to the adequacy of the water supply for the lands within the area to be irrigated. Obviously, if the water supply is ample for the system, and there are no apparent harmful effects from heavy percolation losses, it might be very difficult to create local public sentiment favorable to the expenditure of a large investment for the purpose of materially diminishing the canal losses, either as a matter of public economy, in order that the water thus saved might be available for some other community, or that the irrigable area in the same locality might be considerably enlarged. On the other hand, if the water right furnishes but an insufficient supply for the lands, then the matter of losses from the system becomes one of great moment to the community, and may directly affect the financial success of the enterprise.

Local custom and sentiment are frequently slow to change, and many years will probably elapse before public attention in the west is focused upon the necessity for adequate legislation dealing with this phase of irrigation. Thus far the doctrine of economy in the use of water for this purpose has not penetrated very appreciably into the courts, although some advance in this direction is noticeable. Possibly through further development in irrigation enterprises the enactment of state laws will be secured, to the end that a policy of economy will be gradually and systematically introduced into the operation and management of canal systems.

Many factors affect the problem which confronts the engineer who, in constructing new works, considers the phase of canal losses, and how far he should go in the expenditure of funds to reduce them.

In the design of pumping systems, the matter assumes especial importance, for the cost of a certain portion of the capacity of the plant, and a proportionate share of its operation, up-keep and depreciation, is necessary solely on account of losses; and this may be balanced against the cost of permanent means for reducing them. The promoter is apt to consider expenditures for the purpose of lining or puddling the canals, as unnecessary initially, and would prefer to postpone the solution of the matter to a later date, when its cost could be charged against upkeep. In some cases, this undoubtedly is the best plan to pursue. The value of the products resulting from irrigation is a factor of prime importance in a consideration of the amount of the first cost of the works, for although the efficiency would be improved by lining all of the canals and laterals with concrete, the resultant first cost of the system might be entirely out of proportion to the returns which could reasonably be expected from the land, and this is the source which must eventually be depended upon to repay the investment.

Each case must, therefore, be considered on its own merits, and conclusions drawn accordingly.

EVAPORATION FROM CANALS.

Aside from preventable overflow wastes, the losses from a canal system are usually classed as evaporation and percolation. Of these, the first mentioned is relatively unimportant, and can be entirely eliminated only by carriage of the water in pipes or covered conduits. In localities where such is the practice, in order to conserve the water and prevent percolation, evaporation is, of course, also prevented, and additional saving is thus effected. Ordinarily, however, such construction expense would not be justified for the prevention of the evaporation loss alone.

Other conditions being similar, the loss due to evaporation will vary directly as the area of water surface exposed, and it follows, therefore, that a wide shallow canal section should be avoided, if practicable. Numerous factors enter into the determination of the amount of the loss and render its solution complex and uncertain. The temperature of the water at the surface, the degree of humidity of the atmosphere, the temperature of the air and the action of the wind all affect the amount of evaporation. The loss is naturally greatest during the hottest months of the year, usually from June to August; this is clearly shown in Table I of records obtained in several of the arid states by a series of observations on evaporation from shallow pans set on the ground. The conclusions are, however, not sufficiently accurate to afford other than a general idea of the variation in conditions as they exist in different localities. An average result taken from the table indicates an annual evaporation of about 75 ins.; the maximum being reached in Arizona, and the minimum in Oregon and Nevada. At Boise the annual evaporation amounts to approximately 5 ft. per annum, of which 4 ft. occurs during the irrigation season.

The actual loss from this cause, so far as the average canal system is concerned, is not very significant, as will be apparent when it is considered that for an irrigation season of eight months the average evaporation loss is equivalent to about 5 ft. in depth, or 0.024 cu. ft. per square foot of exposed area per day. This ranges in value from 2 to 5 per cent of the percolation losses, and hence is usually included with the latter in a consideration or determination of the facts for any canal system.

TABLE I.—SHOWING DEPTH OF EVAPORATION, IN INCHES, BY MONTHS.*

Month.	Nebraska.	Idaho.	Oregon.	Nevada.	Washington.	New Mexico.	Arizona.	Average.
January	1.75	1.50	0.50	1.75	1.75	2.50	4.59	2.05
February	1.75	2.25	1.25	1.75	2.50	2.75	4.75	2.43
March	3.00	4.00	3.57	2.25	6.25	4.50	6.25	4.22
April	4.50	7.25	6.64	3.25	7.91	8.00	9.00	6.65
May	6.25	10.68	7.15	5.25	8.36	11.50	11.50	8.67
June	8.05	11.05	6.99	7.86	8.90	13.45	13.50	9.97
July	10.95	11.15	8.01	9.86	10.74	11.57	14.25	10.93
August	9.39	11.77	9.21	8.70	9.41	10.48	14.23	10.46
September	7.44	9.75	6.13	5.13	5.51	8.58	13.76	8.04
October	5.59	5.46	2.50	3.35	3.15	6.76	11.31	5.44
November	4.00	2.70	1.00	2.50	2.00	3.86	7.39	3.35
December	3.00	1.50	0.50	2.00	1.50	3.00	4.65	2.38
Totals	65.67	79.00	53.45	53.65	67.98	86.95	115.18	74.59

*Engineering News, June 16, 1910.

FIELD EVAPORATION.

Although evaporation from the field during irrigation bears only an indirect relation to canal losses, this feature may be worthy of mention here. There is much more considerable loss from evaporation of water during actual irrigation than is experienced in the operation of the canal system alone. Here, too, the systematic determination of the amount of such loss is complicated by many varying conditions, such as to render it very difficult of accurate solution.

The topography of the ground surface, the character of the soil, the amount and kind of vegetation growing thereon, and the methods of irrigating all affect the field evaporation. By the use of the furrow method of irrigation, the loss may be rendered considerably less than by the flooding methods commonly in use, for in the latter case, large areas of water surface are exposed to the action of the sun and wind. Irrigation delivery by the rotation system assists greatly in reducing field evaporation losses, by the accomplishment of the desired result with a larger head of water in a shorter time. Every experienced irrigator is familiar with the sluggishness of the small irrigation stream on a hot sunny day, especially in sandy soil, whereas at night the same stream accomplishes much more satisfactory results. The large rotation head at more infrequent intervals obviates this difficulty to a considerable extent, and frequent cultivation of the soil tends further to reduce evaporation therefrom, and to raise the duty of water.

It is estimated that 20 per cent of the water delivered to the farm ditches is usually lost through field evaporation owing to the use of wasteful methods in handling the supply. In some localities the highly profitable character of the crops raised, coupled with scarcity of water for irrigation, has rendered it not only advisable, but practically necessary, to subirrigate orchards by means of long lines of pipe laid in the soil parallel to the rows of trees. While such practice is feasible from a financial standpoint only under very favorable conditions as to crops and markets, yet it indicates an extreme to which methods can be carried in order to effect an elimination of the field evaporation and other losses.

LOSSES FROM PERCOLATION.

The most serious and important phase of the subject of losses from canal systems with which the irrigation engineer is compelled to deal arises from percolation or absorption of the water within the wetted area of the canal sections. From 30 to 50 per cent or more of the water diverted into the head of the average irrigation canal and distributary system is lost in this manner before it reaches the land and performs any useful function.

An additional amount seeps away from the small ditches of the farmer, before it touches the crop roots, so that even less of the original amount diverted is readily available for the crop. This applies to such systems as are provided with earthen channels for the carriage of the water supply, and it is readily to be seen that the annual total of the amount of water which seeps away is enormous. In this discussion, the canal losses due to evaporation have not been segregated from those due to percolation, for as has been stated the former loss is insignificant compared with the latter.

TABLE II.—SHOWING LOSSES FROM VARIOUS CANALS.

	Name of Canal.	Location.	Length of section tested, miles.	Discharge at head of section, sec. ft.	Percentage lost, per mile.	Loss per sq. ft. of wetted area per day, cu. ft.	Date of test.	Year when constructed, enlarged or first used.	Soil and other conditions.
1.	Moore Ditch	Sacramento Valley, Cal.	0.8	116	9.4	....	July, 1906.	1864	Very gravelly bed.
2.	Moore Ditch	Sacramento Valley, Cal.	4.0	116	1.0	....	July, 1906.	1864	Heavy clay loam, trampled by stock.
3.	Moore Ditch	Sacramento Valley, Cal.	4.0	127	4.1	....	July, 1907.	1864	Same as No. 2—not puddled by stock.
4.	Capay-Winters Canal	Sacramento Valley, Cal.	3.0	79	15.3	....	Aug., 1907.	1903	Wash gravel bottom.
5.	Capay-Winters Canal	Sacramento Valley, Cal.	15.5	43	1.1	....	Aug., 1907.	1903	Clay loam.
6.	Stony Creek	Orland, Cal.	3.3	80	2.17	....	May, 1907.	1888	Canal section much eroded.
7.	Kennewick	Yakima Valley, Wash.	9.0	123.0	2.8	....	Spt., 1904.	1902	Gravelly, porous soil.
8.	Kennewick	Yakima, Valley, Wash.	9.0	79	1.4	....	Oct., 1906.	1902	Same as No. 7, after puddling and silting.
9.	Logan and Richmond	Utah	0.7	59	4.7	2.5	May, 1899.	....	
10.	Wheatland No. 2	Wyoming	17.0	90	1.0	....	July, 1900.	....	
11.	Bear River	Utah	1.5	279	3.3	....	June, 1901.	1889	Heavy rock and disintegrated limestone.
12.	Bear River	Utah	0.75	264	2.5	....	June, 1901.	1889	Porous material.
13.	Bear River	Utah	3.25	258	.34	....	June, 1901.	1889	Sidehill location, heavy soapstone formation.
14.	Grand Valley	Colorado	2.25	260	1.4	....	July, 1901.	1884	Heavy, firm clay and black shale.
15.	Grand Valley (high line)	Colorado	21.25	140	.39	....	July, 1901.	1883	Partly sidehill location.
16.	Lake	Colorado	18	456	.43	....	June, 1901.	1892	Sandy loam.
17.	Sunnyside	Washington	8	612	0.7	1.80	June, 1908.	1892	Volcanic ash.
18.	Sunnyside	Washington	5	451	0.9	1.84	June, 1908.	1892	Sand and volcanic ash.
19.	Sunnyside	Washington	4	41	5.2	2.74	June, 1908.	1892	Porous loam.
20.	Various canals	India	959	...	...	1.00	....	....	Old canals.
21.	Farmers' Union	Idaho	3.3	125	2.09	1.79	June, 1910.	1905	Sidehill, granite and sandwash formation.
22.	Settlers' Canal	Idaho	2.7	151	.42	.44	July, 1909.	1894	Cemented sand and gravel.
23.	Middleton Water Co.	Idaho	1.7	101	1.25	.83	July, 1909.	1876	Clay and silt overlaid with ash.
24.	Ridenbaugh	Idaho	6.8	65	3.1	2.48	July, 1912.	1889	Clay loam underlaid with hardpan.
25.	Ridenbaugh (high line)	Idaho	9.0	43	5.9	2.13	July, 1912.	1910	Heavy lava ash with clay and sand.
26.	Main, U. S. R. S., Boise	Idaho	6.6	617	.085	.12	Ssn., 1912.	1909	Volvanic ash, some sand and rock.
27.	"G" Canal, U. S. R. S., Minidoka	Idaho	18.0	Average 70	1.56	.78	Ssn., 1913.	1908	Clay and lava ash with some gravel
28.	"H" Canal, U. S. R. S., Minidoka	Idaho	22.2	Average 136	.68	.77	Ssn., 1913.	1908 & 1913	Clay and lava ash with some gravel
29.	"J" Canal, U. S. R. S., Minidoka	Idaho	25.0	Average 182	.69	.78	Ssn., 1913.	1908 & 1912	Clay and lava ash with some gravel.
30.	Main North U. S. R. S., Minidoka	Idaho	7.8	Average 912	1.12	1.58	Ssn., 1912.	1907	Rock, sand and gravel.
31.	Main South U. S. R. S., Minidoka	Idaho	13	Average 416	1.14	.92	Ssn., 1912	1911	Rock, sand and clay.
32.	C-2, U. S. R. S., Minidoka	Idaho	1.8	Average 37	2.35	60	Ssn., 1912.	1907	Heavy clay.

In compiling data of this nature, many investigators have expressed the results in feet of depth over the area of water surface in a given time, others use percentages of the total discharge of the canal per mile. A more satisfactory method is to arrange the amount of loss in terms of volume per unit of wetted area for a given time. While the first described methods afford fairly satisfactory means for general comparisons, they take no account of the area exposed to percolation nor of the velocity of flow. The latter method is the more accurate, for percolation loss bears an almost direct relation to the area of wetted perimeter.

Percolation is dependent for its action upon the laws of capillary attraction and of gravity. Its rate, therefore, is a function of the nature of the soil composing the canal section and of its condition as to saturation. Thus, soils of a coarse, granular nature will absorb greatly, until the surrounding locality is so thoroughly saturated that the losses gradually lessen, or until natural silting results in filling the interstices of the soil particles so that the section becomes tight.

Under certain conditions it is not unusual to find gains in the discharges of canals, where side hill locations or underground conditions create opportunities to "pick up" seepage water from adjacent territory. These conditions are far less frequent than those where losses obtain, and are not herein treated. The data shown in Table II are illustrative of the rather wide range of percentages of loss in canal systems at various localities, and surrounded by diverse conditions as to soil, etc. The data were secured from bulletins of the U. S. Department of Agriculture and other sources, and the canal sections upon which the measurements were taken were all unlined. The discharges shown range from 43 sec. ft. to 279 sec. ft. While at first glance it appears difficult to derive any general conclusions from such a range of seepage values as are shown here, especially as the data in regard to the area of wetted perimeter are lacking in many of the cases, yet certain facts are evident

The large percentage of loss per mile in the gravelly or coarse soils as compared with that in the tighter soils is quite marked. It is also plainly evident that the percentage of loss increases rapidly as the discharge of the canal decreases, other conditions being similar Long usage and seasoning of the canal in many cases have resulted in a decrease of percolation losses, especially if the water supply has carried enough silt to effect a partial closure of the pores of the soil and if this silted layer has not been disturbed by cleaning processes. In other cases, however, where the velocity of the discharge has been too great to permit of the settling of silt, scour has prevented such beneficial results, and such old canals probably leak as badly as when first constructed. Lack of silt in the water will also delay the natural curing of the trouble.

A case in point is the system of the gravity unit canals on the Minidoka Project of the U. S. Reclamation Service, in Idaho. These canals were first operated in 1907, and have been used each season since then. The topography of the project is such that it is necessary to check the flow of the water at frequent intervals in order to effect diversion at the proper height, and this lowers the velocities, thus increasing the percentages of loss by percolation. The soil of which the canal sections are formed is in many cases almost pure sand and of a coarse nature. The water from Snake River which supplies the system is usually quite free from silt or any foreign material, and such as may be suspended therein is afforded opportunity for being deposited in Lake Walcott, which acts as a settling basin above the diversion dam. As a result, the canals have received practically no silt deposit, and the leakage is quite high in many of them, even after seven years of service In fact, largely as a result of the percolation from one canal of 180 sec. ft. capacity, the surrounding land for probably a quarter of a mile on either side became unfit for cultivation, although it has never been irrigated. In 1912 this canal seeped to a greater degree than had ever before been known, although that was its sixth season of use, and it has never been cleaned out or disturbed. In 1913, there was a slight amount of very fine silt carried in the Snake River for several weeks, and this found its way into the irrigation system The canal in question tightened up, the evil effects of the seepage were markedly less, and the adjacent land dried out to a considerable extent. It may be added that local beds of clay are not available for puddling or silting, and the problem of lessening the percolation loss is one for which an inexpensive solution has not been found.

In Table III there are compiled data from several of the projects of the U. S. Reclamation Service, covering specific cases of unlined smaller canals or laterals, with discharges of from 1.95 sec. ft. to 27.7 sec. ft. Here again the effect of the loose porous soil is shown as also the fact that for the smaller sized laterals the values of the percolation loss are greater than where the discharge is considerable. In this table the average loss in cubic feet per square foot of wetted area per day is seen to range from 0.22 in the case of a mud section underlaid with hardpan to an extremely high value of 5.13, where the material is a coarse blow sand and where there is very little velocity in the lateral on account of the necessity for checking the flow. The average range of the values is from 1.0 to 2.0 cubic feet.

TABLE III.—SHOWING LOSSES IN LATERALS ON THREE PROJECTS OF THE U. S. RECLAMATION SERVICE.

	Name of Project.	State.	Length at head of section tested, miles.	Discharge of section, sec. ft.	Percentage lost, per mile.	Loss per sq. ft. of wetted area per day, cu. ft.	Date of test.	Year when constructed, enlarged or first used.	Soil and other conditions.
1.	Sunnyside (various laterals)	Washington	5.0	1.95 to 4.05	13.6	1.4	Ssn., 1908.	....	Sand.
2.	Boise (Mora)	Idaho	2.1	9.51	2.52	.22	July, 1909.	1904	Mud, underlaid with hardpan.
3.	Boise (Teed)	Idaho	2.0	11.20	1.34	1.39	July, 1909.	1909	Mud, underlaid with hardpan.
4.	Boise (Kennedy)	Idaho	5.8	20.43	.37	.68	July, 1910.	1910	Mud, underlaid with hardpan.
5.	Boise (Eight Mile)	Idaho	5.7	32.3	2.32	1.18	July, 1910.	1910	Mud, underlaid with hardpan
6.	Boise (Partridge)	Idaho	2.7	6.0	3.55	1.13	July, 1912.	1911	Volcanic ash, underlaid with hardpan.
7.	Boise (South Nampa)	Idaho	2.4	17.0	6.2	1.27	July, 1912.	1910	Volcanic ash and adobe, underlaid with hardpan.
8.	Boise (Deer Flat, low line)	Idaho	19.0	27.7	2.15	1.19	Ssn., 1912.	1909	Sand and volcanic ash. Sidehill.
9.	Minidoka (210-2)	Idaho	1.5	12.3	1.26	2.56	Ssn., 1913.	1913	Sand underlaid with fine gravel.
10.	Minidoka (125)	Idaho	0.8	6.9	40.00	5.13	Ssn., 1913.	1913	Blow sand.
11.	Minidoka (97)	Idaho	0.8	16.6	35.0	4.73	Ssn., 1913.	1912	Sand and sandy loam.
12.	Minidoka (185)	Idaho	3.2	7.68	15.7	2.00	Ssn., 1913.	1912	Clay and sand.
13.	Minidoka (1011)	Idaho	1.8	9.82	23.9	2.02	Ssn., 1913.	1913	Clay and sand.
14.	Minidoka (423)	Idaho	3.8	3.50	24.4	1.12	Ssn., 1913.	1912	Sand and sandy loam.
15.	Minidoka (G-15)	Idaho	4.4	6.2	5.0	.75	Ssn., 1913.	1909	Lava ash.
16.	Minidoka (G-19)	Idaho	2.7	3.0	17.25	1.53	Ssn., 1913.	1909	Sandy.
17.	Minidoka (J-19)	Idaho	2.4	4.4	9.15	1.04	Ssn., 1913.	1909	Lava ash and clay.

From a large number of experiments conducted by the Department of Agriculture the following results forcefully indicate the effect of the capacity of the canal upon the loss due to percolation. As these investigations covered a wide range of conditions, the figures may be assumed to be fairly representative:

	Percent loss per mile.
Canals carrying 100 sec. ft. or more	0.95
Canals carrying 50 to 100 sec. ft....	2.58
Canals carrying 25 to 50 sec. ft.....	4.21
Canals carrying less than 25 sec. ft..	11.28

Compiling the results of observations on several hundred miles of canals in eight different projects of the U. S. Reclamation Service, Mr. E. A. Moritz has prepared a list of average values for losses in unlined canals, constructed in various types of soil.* The data agree closely with the results indicated in the foregoing tables.

Kind of material	Loss in cu. ft. per sq. ft. of wetted area per day.
Cement, gravel and hardpan with sandy loam	0.34
Clay and clay loam	0.41
Sandy loam	0.66
Volcanic ash	0.68
Volcanic ash with some sand	0.98
Sand and volcanic ash or clay	1.20
Sandy soil with some rock	1.68
Sandy and gravelly soil	2.20

As these canals were generally those which had been in use from three to six years, it appears that there may be expected in an unlined earth canal system a percolation loss of between 0.40 cu. ft. and 2.5 cu. ft. per day for each square foot of wetted perimeter. In cases of extremely porous material, however, this maximum figure may be exceeded by 100 per cent as has been shown in Table III. The minimum value will not usually be less than 0.4 or 0.5 cu. ft. even in practically impervious material.

In the design of the canal system the necessity for providing sufficient capacity to care for the losses should be carefully considered, and the relative permeability of the soils to be traversed should be determined, after which the percolation losses may be closely calculated. Mr. E. A. Moritz has proposed a formula for the determination in second feet per mile of the amount of seepage loss in any canal, where sufficient data is available to permit of assuming a loss in depth per 24 hours over the wetted area. This formula is as follows:

$$S = 0.2c\frac{Q^{1/2}}{V^{1/2}}$$

where S = seepage loss in second feet per mile of canal.

c = depth of loss in feet over wetted area in 24 hours.

Q = discharge of the canal in second-feet.

V = velocity of water in feet per second.

The author of this formula eliminates the factors of variation in canal section as being non-essential within ordinary limits, and thus involves only three variables in the equation, rendering it simple of application to the usual problems of this nature. It is quite apparent from a casual study of the formula that velocities should be as large as practicable, in order to reduce the percentage of percolation loss.

In any given canal system the total losses due to evaporation and percolation where no special preventive measures have been adopted will reach considerable proportions. While the rate of loss in depth per 24 hours affords a proper and scientific method of studying the problem, it does not, without further computation, give an adequate idea of the value which these losses assume in terms of the total amount of water diverted into the system. In order to show this feature more strongly Table IV has been prepared from data covering conditions on eight of the projects of the U. S. Reclamation Service. From this table it is apparent that from 17 to 45 per cent of the water which was diverted at the headgates was lost before it was diverted from the canals into the distribution systems. For the purposes of this discussion, a distinction is here made between the canals or larger waterways and the laterals or distribution systems with their branches.

Careful measurements on the South Side Pumping Unit of the Minidoka Project, Idaho, where the lateral system, covering 48,000 acres, has been in use for four years, show that in 1912 27 per cent and in 1913 22 per cent of the water diverted into the sublateral system was lost. On the Yakima, Washington, project in 1910 32 per cent of the water taken into the lateral system was lost, and on the North Platte, Nebraska project, the losses in some portions of the lateral system reached a maximum of 39 per cent. Even though successive years of usage may serve to reduce the heavy percentage of percolation, it is clear that with age comes the necessity for cleaning, especially in the case of small ditches, and thus the top layer of silted soil is removed, exposing the more open soil below. In an unlined or untreated system, therefore, the loss is likely to be considerable as long as it is used. On the Ganges Canal in India, several hundred miles in length, the absorption has reached as high as 70 per cent in some years. On some private projects in Utah and Wyoming, the canal losses range from 28 to 63 per cent. In one case 45 per cent was lost in a distance of 29 miles.* In general it may be said that losses of from 30 to 50 per cent are common in unprotected earth systems, both in this and in older irrigated countries. The conditions surrounding any particular case should be carefully investigated before any conclusions can be successfully drawn, however, as the factors entering into the result are many and varied.

Table V has been prepared to show in detail the effect on losses from a canal system, taken as a whole, caused by the varying monthly requirements throughout the irrigation season. The data show the record of such losses for the South Side Pumping Unit of the Minidoka Project for the season of 1913. The soils throughout the tract are fairly uniform in character, so that the figures should be quite typical of the lava ash soils common to many of the arid states. None of the canals or laterals are lined or treated; velocities of flow do not exceed 2.5 to 3.0 ft. per second, as a rule, and the laterals are in many cases quite small, inasmuch as deliveries are carried to each farm unit; the system has been operated for four years.

There is a very noticeable peak demand during July and August, when the canals are run very much more nearly to capacity than either earlier or later in the season. Nevertheless, it is necessary, owing to the topography of the tract, to check the flow to normal height in the laterals as well as in the canals at frequent intervals to effect satisfactory deliveries. This has its effect on the losses in a noticeable manner. An examination of the table will show, for instance, that of the total amount of water diverted into the lateral system in May, 18.2 per cent was lost, while in July, one of the months of greatest demand, only 15.6 per cent was lost, thus indicating that to reduce percolation, it may be best to operate the ditches to full capacity for a time and then turn the water out. In this instance, it has been customary to operate the canals continuously during the season but to rotate the deliveries of large heads between the water users. The table also indicates plainly that the percentage of losses in the entire system expressed in terms of the amounts pumped in each month is at a maximum at the beginning and at the end of the season and at a minimum in July; merely additional evidence that the discharge and the velocity are very essential factors in regulating the percolation loss of an unlined canal system.

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*Engineering News, Aug. 28, 1913.

*Discussion on Use of Water in Irrigation by Elwood Mead.

TABLE IV.—SHOWING COLLECTIVE LOSSES IN CANAL SYSTEMS ON EIGHT PROJECTS OF THE U. S. RECLAMATION SERVICE.

Project.	Year.	Percent of amount diverted, delivered to laterals or wasted.	Percent of amount diverted, delivered to land or wasted	Per cent of amount diverted, lost in canal system.	Soil conditions.
Umatilla	....	68	..	32	Sandy loam and volcanic ash.
Truckee-Carson	....	59	..	41	Sand, sandy loam and clay.
Okanogan	....	55	..	45	Volcanic ash and gravel.
Orland	....	77	..	23	Sandy and gravelly loam, silt loam.
Klamath	....	52	..	48	Decomposed basalt.
Tieton	....	76	..	24	Sandy loam, gravel and volcanic ash.
Sunnyside	1909	85	..	17	Sand and volcanic ash.
Minidoka, north side	1910	71	..	29	Sand and sandy loam.
Minidoka, north side	1912	72	..	28	Sand and sandy loam.
Minidoka pumping unit.	1911	..	69	*31	Lava ash.
Minidoka pumping unit.	1912	..	66	*34	Lava ash.

*This includes losses in the lateral system also.

In a discussion of the "Economic Aspect of Seepage Losses in Irrigation Systems" (Proceedings American Society of Civil Engineers, December, 1912), Mr. E. G. Hopson has pointed to the value of such losses to the community and has presented valuable data, some of which has already been quoted in Table IV. As above stated, the problem of how far methods should be pursued in any individual case towards the reduction of the canal losses is one that must be solved on its own merits; no general rule can be given. If the value of the water to a community is not yet very important owing to the low returns from crops produced, and if the percolation appears unlikely to seep adjacent lands, it would perhaps be an unwise investment to resort to extreme methods for saving the water, until further developments more nearly justified the outlay. There are undoubtedly many localities, however, where the matter should be very seriously considered from a sound financial basis, and it is believed that such consideration would show that preventive measures would pay.

METHODS OF LOSS PREVENTION.

One of the most common methods of reducing losses by percolation in canals consists in the introduction of qualities of finely divided clay into the water. If the water naturally carries silt in suspension, and the velocity of flow is moderate enough to permit of the settling of the suspended matter, the result will be beneficial and a few seasons of use will tighten the upper layer of the canal sections. Where there is no silt in the source of supply it may be placed in the water above the points where seepage conditions are the worst, and permitted to travel with the current, the water being agitated, if necessary, to overcome a tendency toward too early settling.

On the Minidoka project, as has already been stated, the losses by percolation from the canals and the distributing system north of the Snake River have been very large. This has contributed in some measure to the conditions which brought about the necessity for drainage works and these works are now under construction. Except in the westerly portion of the project, where the soil is a heavy clay, there are practically no deposits of suitable material for locally treating the canals in order to tighten them by silting. The broad, shallow sections made necessary by the topography render any resort to lining not feasible, and other conditions have also made this impracticable. Adjacent to the upper end of the main canal, however, there is an area of fine clay, which it is believed will be suitable for silting the canal system, and plans are being made to accomplish this result as a preventive feature of the drainage work. It is planned to loosen the material with giants and sluice it to a pumping plant, from which it will be carried to the canal s edge and deposited in the water. It has been roughly estimated that 50,000 cu. yds. of material will be required for this work, but details are still to be perfected. As the velocities in the canals are in nearly all cases necessarily low, the accomplishment of the plan as a far-reaching means of tightening the canals may be attended with some difficulty.

In the article quoted above Mr. Moritz relates that a very porous canal 1½ miles in length was puddled and made practically tight by throwing finely pulverized material into the water, and having it worked into the canal bottom and sides by men wading with rubber boots. On the canal in question, which was 6 ft. wide on the bottom and 3.7 ft. deep, the work was accomplished for about $0.10 per square yard.

Another method of reducing percolation losses in local areas of canal section, and one which has long been in successful use, consists in blanketing and puddling the bottom and sides with a more impervious material than that through which the canal was constructed. On the Kennewick Canal, in the Yakima Valley, Washington, the gravelly, porous soil was excavated from the bottom for a foot or so below normal grade, and this was replaced by moist clay, tamped into place. The bottom and sides of the canal were then plastered with several inches of soft clay mortar, and the whole was then covered with gravel to prevent washing. The results were quite satisfactory. It is frequently difficult, however, to locate suitable clay deposits which are close enough to render the work economical.

TABLE VI.—COST OF LINING IRRIGATION CANALS.

Description of lining.	percent of water saved.	Experimental cost of lining per sq ft., cents.
Earth, no lining	0	....
Cement—concrete 3 ins. thick	86.6	8.30
Cement lime concrete, 3 ins. thick	65.5	8.30
Cement mortar	63.3	3.88
Heavy oil, 3⅔ gals. per sq yd.	50.4	1.20
Clay puddle, 3.5 ins. thick	47.8	3.90
Heavy oil, 3 gals. per sq. yd.	38.0	1.00
Heavy oil, 2⅓ gals. per sq. yd.	27.3	0.77
Thin oil, 2.5 gals. per sq. yd.	7.3	1.00

TABLE V.—SHOWING, IN PERCENTAGES, THE AMOUNTS DIVERTED INTO AND LOST FROM THE SOUTH SIDE PUMPING STATION UNIT, MINIDOKA PROJECT, IDAHO, SEASON OF 1913.

Month.	Quantities in percent of totals for season			Quantities in percent of amounts pumped during the month shown		Loss in lateral system in percent of amount diverted into laterals during month shown.
	Amount pumped.	Loss in main canals.	Loss in lateral system	Loss in main canals.	Loss in entire system	
May	13.6	18.0	9.5	26.1	26.9	18.2
June	18.6	11.3	22.0	15.0	33.2	27.0
July	24.0	25.8	18.6	18.6	30.6	15.6
August	25.2	26.2	27.4	20.4	37.2	22.7
September	17.6	17.8	20.2	19.8	37.6	27.0
October	1.0	0.9	2.3	*	*	*
Total, or mean	100.0	100.0	100.0	19.6	35.1	22.2

This tract covers about 18,000 acres, and is supplied from three pumping stations. The soil is principally lava ash, with some heavier clays and a small area of sandy soil. The subsoils are gravelly, with small areas of hardpan.

*No data for October shown; water was run only 4 days.

As affording an interesting comparison of the cost and primary results in saving of percolation losses, due to different methods and materials used for lining, the data given in Table VI, from experiments conducted by the U. S. Department of Agriculture in conjunction with the University of California, are instructive (Annual Report for 1907, Office of Experiment Stations, U. S. Dept. of Agriculture). The costs do not include preparation of the ditch.

It appears from these experiments that the use of heavy oil for treating the wetted section of the canal is, next to a mortar or concrete lining, the most effective method of reducing losses by percolation. The oil treatment, however, is not permanent, and after a season or two must be repeated if the good effects are to be continued. One of its advantages lies in its prevention of the growth of weeds and other vegetable matter within the area treated. The use of any cost data for purposes of estimates on similar work should be attended with caution, except where full details are available, and the figures herein shown are for purposes of general comparison only.

Where concrete linings have been placed in large canals it has, with but few exceptions, been done after the system had been in use for some years, and had either seeped so badly as to justify the expense of lining or else required additional capacity, which could be obtained by reducing the coefficient of friction with concrete. In the latter case, of course, both of the beneficial results would follow. If it can be demonstrated that the installation of the concrete lining is justified at the time of initial construction this would be the logical time for it to be placed, as a smaller section could then be excavated and steeper side and bottom slopes provided, resulting in greater velocities, with less friction, in addition to the saving of the seepage loss and the cost of numerous drops and checks where the slope of the ground surface is considerable.

In the article of Mr. Hopson above quoted the saving of percolation loss as a result of lining a lateral on the Umatilla project, Oregon, was sufficient to justify the conclusion that these losses could be reduced to less than 10 per cent of the total amount of water diverted, if the system were lined in its entirety; unlined the losses were 50 per cent. In the Gage Canal, near Riverside, California, the losses amount to but 6 per cent, due to the cement lining.

The cost of the thin cement mortar linings used on several miles of the small laterals of the Umatilla project varied from $0.489 to $0.764 per square yard, including overhead charges. The lining was 1½ ins to 2 ins. thick, and much of it was placed during cold weather. It was trowelled by hand, but mostly mixed by machine. The results, as stated above, were very satisfactory. Similar work was accomplished by Mr. Etcheverry at about the same cost. It would appear that such thin linings would be unsuitable for severe climates where heavy frost conditions might tend to crack and heave the mortar. Excellent drainage conditions in the soil beneath the lining would, however, doubtless greatly assist in preventing this. The removal of accumulations of blow sand or trash from a lateral lined as described would require to be handled with considerable care, and possibly at greater expense than with an unlined section.

On the Boise, Idaho, Project of the U. S. Reclamation Service, the main canal was lined for nearly six miles, with a 1:3:6 mixture of concrete, at a cost of less than $0.10 per square foot. The lining was 4 ins. thick; the canal section 40 ft. on the bottom, and 8 ft. deep, with 1½ to 1 side slopes. In this instance, the work was done to provide additional capacity without enlargement, and also to reduce the danger from breaks, as well as to eliminate percolation losses.

Timber and galvanized metal flumes when used as part of a distributing system serve several purposes, and not only carry a greater discharge for a given area of section, but if properly built and maintained should exhibit practically no seepage loss. The same is true of pipe lines. On the Umatilla and Yakima projects of the U. S. Reclamation Service, considerable lengths of concrete pipe have been installed, and operated under various heads. Tests showed that the seepage loss was almost directly proportional to the head on the pipe, and that of 4.4 miles of 4-ft. pipe operated under heads of from 19 ft. to 85 ft., the seepage loss per mile ranged from 0.04 sec. ft. to 0.20 sec. ft. or about 0.021 sec. ft. per mile for each 10 ft. of head. The use of pipes and flumes for crossing draws and coulees also frequently saves in the length of the line, and by avoiding long detours this prevents the exposure of the system to that additional extent.

There are certain practices of an ordinary nature that may be avoided with but little additional expense, to the benefit of the newly constructed canal system. For instance, it is not unusual in the planning of such works, especially where economy in expenditures is necessary, to omit the upper bank of the canal when crossing a small draw, or on sidehill locations. This results in exposing unnecessarily large areas to absorption and to that extent is a wasteful method. The least exposed area of water surface, coupled with the shortest length of wetted perimeter and the greatest practicable velocity are, from this standpoint, the desired ends to be secured in the design of the system and whenever the more fundamental principles will permit, these features should be kept well in the foreground. Careful attention during construction to the bond between the natural ground surface and the canal embankments, as well as to the necessity for thorough rolling or compacting by other means of the layers of soil composing the banks, may result in a considerable reduction of seepage loss. It is also desirable that the material in the banks shall be disposed of in such a way that the coarser soils shall be deposited on the outside.

The use of the rotation method of delivery of water has already been touched upon, as desirable for reducing evaporation losses in the field of the irrigator. It is likewise valuable as a method for conserving the canal discharges and reducing the seepage losses. On some projects, it is found to be feasible to run water in the canal system for a period of ten days to two weeks, after which a like period will ensue during which the entire system is dry. The advantage of this in the reduction of losses is obvious. Other systems of rotation provide for carriage of water at stated intervals in the laterals only, or in one part of a lateral while the other portion is empty; still other methods keep the distributing system full continuously, rotating the supply between the various water users.

Any method which will leave a portion or all of the canal system dry for a portion of the time may be beneficial so far as the saving in losses is concerned. It has been found on the other hand that in some cases the alternate drying and wetting of the silt deposit formed on the slopes and bottom of the canal cracks and shingles it, with the result that the losses are apt to be greater than if the canal had been kept full during the entire season. Timber flumes and other structures may also suffer greater deterioration under a system of periodic flow than when kept wet. The particular method to be adopted, therefore, should be chosen only after full consideration of the requirements of each case. However, the service of large heads of water to the individual users at intervals will usually effect a saving of water in the unlined farm laterals, regardless of the method adopted in the operation of the canals, and should be encouraged if for no other reason than this. The use of the large head of water properly handled will also save a large part of that which is usually lost by percolation in the field when rotation is not in effect. This feature also has a direct bearing on the evil effects of water-logging the land due to the use of too great quantities of water for irrigation. There can be no question but that where it is at all practicable the rotation method of delivery is to be prefered to other methods.

TABLE VII.—DATA FROM EXPERIMENTS ON THE BOISE PROJECT, IDAHO, IN 1910, SHOWING SURFACE RUNOFF FROM IRRIGATION.

Acreage in plat.	Nature of crop.	Soil conditions.	Amount of water used per acre in crop exclusive of runoff, acre ft.	Percentage of runoff or waste in terms of entire amount of water applied to land.
135	Mostly alfalfa, timothy and clover............	Lava ash and sand underlaid with gravel	4.07	11.4
14.3	Alfalfa	Lava ash and sand underlaid with gravel	3.37	6.8
12.1	Wheat	Lava ash, sand and adobe.	.91	28.4
8.2	Alfalfa	Lava ash and adobe underlaid with hardpan.......	2.99	18.5

WASTE WATER.

In the operation of a large system of canals, there are certain conditions to be met in order that proper service may be effected. Where the topography requires that the water shall be delivered at all times during the irrigation season, at the normal elevation or full height, the system must either be provided with a series of checks or there must be a certain amount of water run into the canals during the early and late portions of the season, over and above what is required for crops, and this must be wasted back into some convenient channel, and returned to the river. There is probably but little difference in the amount of percolation loss by either method; the condition is an unfortunate one in that it continually exposes a large area of wetted perimeter to absorption.

It is generally recognized also that in nearly all large and delicately adjusted systems of canals and distributaries it is advisable to flow a little too much rather than too little water in the main canals for the supply of the various branches. In other words, operating conditions can rarely be adjusted with such nicety that all demands, including those of seepage, are met to the exact amount required, and there should be wasteways or spillways where the slight surplus can be cared for, preferably in such a way that it may be rediverted into another canal or lateral lower in the system. A third source of waste comes from the field of the irrigator. Even with careful attention the experienced farmer is frequently apt to find that there is a small surplus or runoff at the lower end of his field, when his ground has a good slope, and the furrows are steep; and when the flooding method of irrigation is used there is also apt to be a surplus of water at the lower end of the field unless considerable care is exercised in handling the irrigation. Within reason, and under some conditions, this too may be classed as unpreventable waste, although it not infrequently happens that a portion of such waste water can be picked up into a lateral and used again for irrigation at a lower elevation.

On the Boise Project, Idaho, during the irrigation season of 1910, a series of measurements was made by the Reclamation Service to determine the actual duty of water on the land. Among other data, measurements were taken of the waste water from the various small tracts under investigation. The results are shown in Table VII. The method of irrigation used was almost exclusively the flooding method and the irrigators were permitted to use all of the water they desired, and to apply it in accordance with their own ideas and practices. It affords an idea of how the average farmer will irrigate on ground fairly well suited to flooding, and in a locality where water is not so valuable as to exact the greatest care in its use.

It may be observed that the percentage of runoff varies inversely with the amount of water applied to the field. Probably from 10 to 15 per cent would be a fairer average of the surface runoff or waste from the irrigated lands. There is no doubt but that this amount could and should be reduced very considerably to the end that economy may prevail in the use of water for irrigation.

Water Works

Use of Compressed Air for Mixing and Placing the Concrete Lining for the 8-ft. Rock Tunnel Intake of the St. Louis Water Works.

(STAFF ARTICLE.)

The designs for the new water works tunnel and intake tower of the St. Louis water works were described and illustrated in an article published in this journal of Oct. 29, 1913. An article describing the contractor's plant and methods employed in driving this tunnel through rock was published in the issue of May 6, 1914. The present article describes the plant and methods employed in placing the concrete lining for this rock tunnel by means of the compressed air system. The finished tunnel will be 8 ft. in clear diameter and about 2,700 ft. long. The concrete lining varies in thickness from 9 to 18 ins. It is not reinforced. The tunnel extends 2,200 ft. under the Mississippi River, at the Chain of Rocks, and the land section is 500 ft. long. The shore shaft is 96 ft. deep.

The pneumatic mixer and conveyor was set at the bottom of the shore shaft, as shown in Fig. 1. Arrangements were made to load the mixer from the top of the shaft by building a measuring hopper at the shaft top with an 8-in. pipe leading down and discharging into the mixer. The hopper at the top was made with a sliding door at its bottom, so that upon signal from the man at the mixer, this door could be opened, thus allowing the measured batch to drop through an 8-in. pipe into the mixer. The man who operates this door also lets in water from a barrel at the same time that the batch is dropped. The water is either fed into the 8-in. pipe at the top of the shaft or is discharged directly into the mixer hopper at the bottom of the shaft through a 2-in. pipe provided for that purpose. This man also unties the cement bags and puts the proper amount of cement into the hopper. At the bottom of the 8-in. pipe above mentioned a trough or chute about 4 ft. long, lined with metal, was placed with a slope of 15°. This chute was placed to break the fall of the concrete materials.

The view shown in Fig. 1 clearly indicates the construction of the mixer. Air was piped down the shaft through a 4-in. pipe. This pipe was reduced to 3 ins. near the lower end and the three smaller air pipes shown entering the machine from the right-hand side of the picture are fed from this 3-in. main air pipe. The lower air pipe is 2 ins. in diameter and, as shown, it discharges directly into the machine end of the discharge pipe to transport the concrete to place in the forms. The upper air pipe is 1½ ins. in diameter and, as shown, discharges at a height in the mixer sufficient to mix the concrete and to aid in transporting it. The small pipe supplies air to the small cylinder shown on the side of the machine. The function of this cylinder is to close a gate at the top of the mixer when the charge of concrete materials has entered the mixer. This gate, of course, is provided to prevent the escape of air from the mixer top when the transporting or mixing blasts are applied. The several air pipes are placed in service at the will of the operator to control the sequence of charging, mixing, and transporting operations.

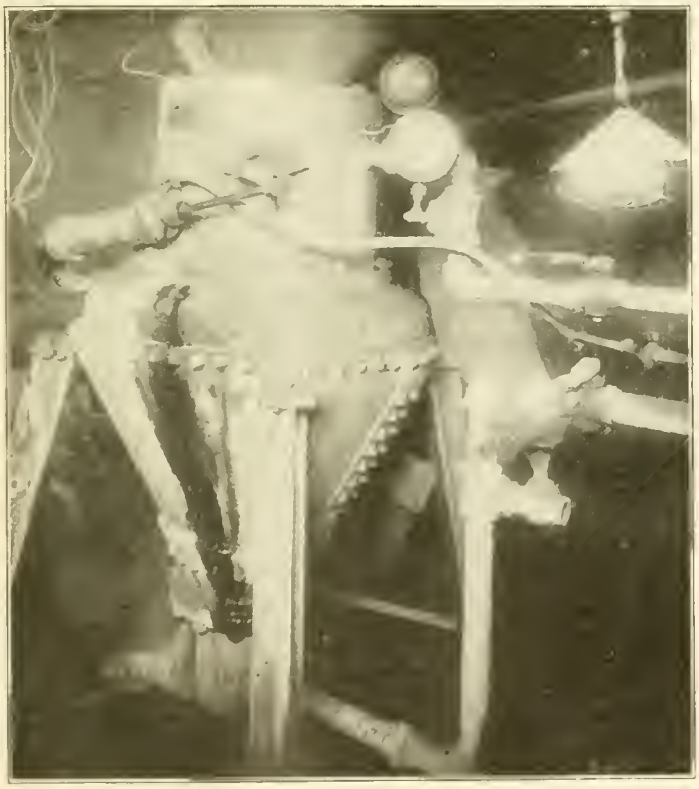
Fig. 1. View of Compressed Air Concrete Mixing and Placing Machine Employed in Placing Concrete Lining of 8-ft. Water Works Intake Tunnel at St. Louis.

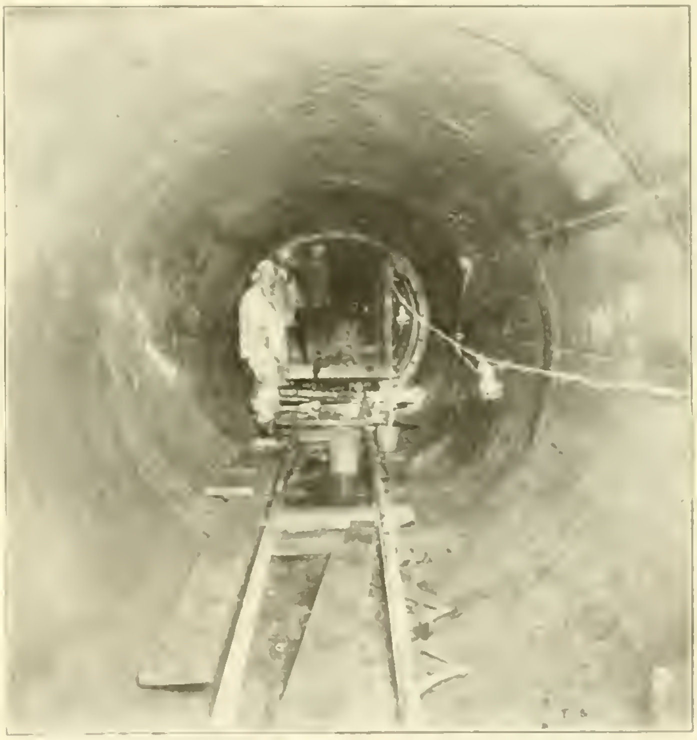
Fig. 2. View Showing Portion of Completed Lining of St. Louis Water Works Tunnel as Placed by Compressed Air Method. Construction and Method of Moving Forms Also Shown.

The land section of the tunnel was first concreted. Work was begun with a line of 8 in. discharge pipe about 170 ft long extending along the tunnel to the forms. The forms are built of structural angles and channels with wooden lagging and light sheet metal nailed to the lagging, as shown in Fig. 2. The forms were portable and were carried on two small trucks running on a narrow gage track placed on the invert. The forms are 36 ft. long and are provided with screw jacks built on the trucks for lowering and raising, and with turnbuckles which are used to pull the wings in laterally to provide clearance when moving the forms.

The forms were first set up about 50 ft. from the end of the tunnel and the pipe from the mixer was run through them to carry concrete for placing the invert. After this section of invert was placed the forms were pushed back over the completed invert and were made ready for concreting the arch. A heavy bulkhead, placed at the rear, was reinforced with sand bags. The concrete discharge pipe was laid along the tunnel from the machine to within 20 ft. of the forms and here a 22½° elbow was inserted and then a 20-ft. length of pipe was added, which brought the pipe up to the roof of the tunnel, where another 22½° elbow was placed in the line, then a straight piece of pipe about 18 ft. long was added, which carried the end of the pipe over the top of the form to about the middle point of its length. The front bulkhead was made by bolting a 9-in. board to the forms and then, as the concrete began to fill up, sand bags were laid to complete the bulkhead and to fill up the irregularities between the 9-in. board and the rock.

The concrete in discharging from the pipe at a velocity of about 75 to 100 ft per second is discharged against the back end of the arch, where it flows down over both sides and assumes about a 30° angle. When the form is filled so that the arch is within a foot or so of the pipe, the last section of the pipe is removed so that the end of the pipe is then projecting just through the bulkhead and the balance of the section is completed. The concrete is blown in until the concrete is within a foot of the pipe and this last portion of the arch may be filled nearly full by discharging a small batch of the proper volume. The usual practice, however, is to move the forms forward until it just overlaps this little cavity.

The first 500 ft of tunnel was started April 27 and was completed May 22. The intention at first was to run one arch every other day or three arches a week, and to run invert on the intermediate days, thus allowing the arch to stand 36 hours. It was found, however, that the arch could be moved every day, so that it resulted in running four arches in four days and the other two days were devoted to running invert, etc. On the days for running the arch the form was moved in the morning and made ready to begin concreting at 1 o'clock in the afternoon, as it was found that the arch was always completed before 5 p. m., and on one occasion the arch was run in three hours.

The compressor plant for this work was originally provided for the excavation of the tunnel and consists of one 950 cu. ft. Ingersol-Rand straight line machine and one 678 cu. ft. Norwalk tandem compound machine. There are three air tanks on the work, so there was an opportunity to determine by experiment how much air was required to operate the pneumatic mixer. To determine the amount of air used all three of the tanks were first filled to 80 lbs. pressure. The compressor was then shut down and a ¼ cu. yd batch was discharged. The combined space in the tanks was 349 cu. ft. Two of the tanks are 125 cu. ft. capacity each and one is 99 cu. ft. The second batch was discharged by using one of the large tanks and a small one and the third batch was discharged using one of the 125 cu. ft. tanks. The amount of air used calculated by reading the pressure gage after the first shot was 1.7 cu. ft. of free air per lineal foot of pipe when using the three tanks and wa
1.4 cu. ft. of free air per lineal foot of pip
when using two tanks and was 1.1. cu. ft. o
free air when using a single tank. This show
that the amount of air used may be kept dow
by using the right size air receiver and, fro
calculations made as a result of this exper
ment, it shows that the size tank required
one which contains 30 cu. ft. of space f
each 100 ft. of pipe through which the con
crete is to be transported.

The work on the St Louis water works tu
nel is now being started on the section unde
the river. The mixer has been moved to
point 1,000 ft from the end of the tunnel an
is placed in a hole about 4 ft deep, so th
the material for concreting may be dropp
into it from a car into the mixer which is
the level of the track. Two forms have be
set up, one at the end of the tunnel 1,000
from the machine and the other form is 5
ft from the machine. The invert for th
section has already been placed so that th
arch forms will be run on alternate days. T
pipe line from the machine is elevated abo
3 ft. above the invert so that it passes throu

the first set of forms and will not have to be moved, as this set of forms approaches the mixer.

The Fruin-Colnon Contracting Co. of St. Louis are the general contractors on the construction of the new tunnel and intake tower. Mr. A. P. Greensfelder is secretary and general superintendent for the company and Mr. J. E. Vollmar is contractor's superintendent in charge. The work is being carried out for the water department of the city of St. Louis. Mr. E. E. Wall is water commissioner, Mr. Gurdon G. Black is chief engineer of the supply and purifying division, and Mr. E. C. Davis is resident engineer in charge for the city. The compressed air concreting apparatus here illustrated and described was leased to the general contractor by the Concrete Mixing & Placing Co., of 123 W. Madison St., Chicago. We are indebted to the latter company for the information from which this article has been prepared.

Condition of Inside of 48-In. Steel and Cast Iron Pipe After 17 Years' Service, New Bedford, Mass., Waterworks.

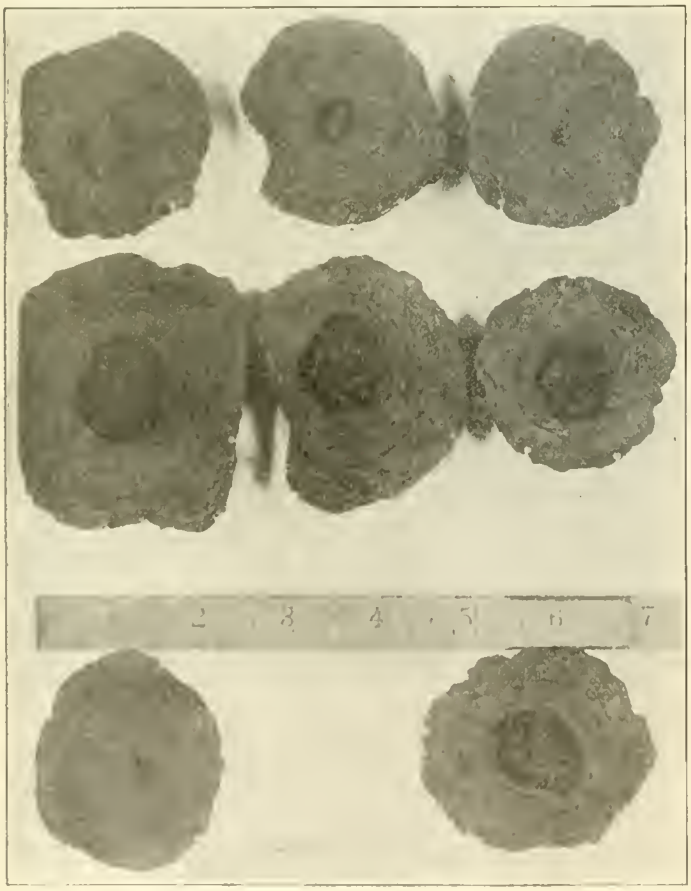

Fig. 1. Rear View of Tubercules Taken From 48-in. Cast Iron Water Pipe at New Bedford, Mass.

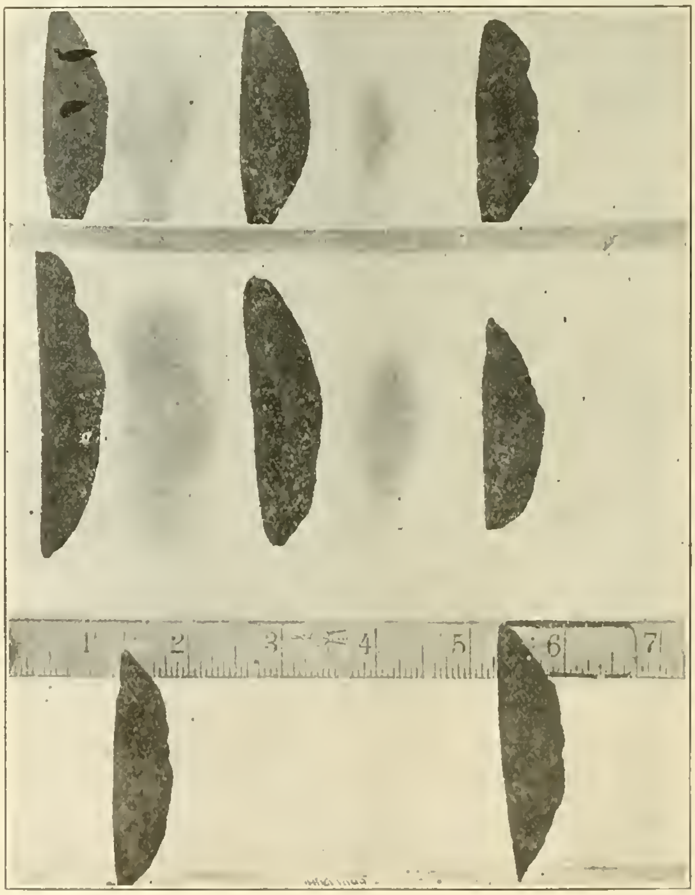

Fig. 2. Side View of Tubercules Taken From 48-in. Cast Iron Water Pipe at New Bedford, Mass.

In May of 1913 an inspection was made at New Bedford, Mass., of the external and internal condition of the 48-in. steel and cast iron force-mains of the city's waterworks system. The investigation was made by Mr. Thaddeus Merriman of the engineering staff of the Board of Water Supply of New York City, Mr Leonard Metcalf, Mr. George H. Nye, city engineer, and Mr. R. C. P. Coggeshall, superintendent of the New Bedford water department. The results of the inspection are summarized in Mr. Coggeshall's annual report for 1913. The present article is based on that report.

The party first inspected the "High Hill" Section. This pipe was installed in 1896 and had been in service 17 years at the time of the inspection here reported. It was last inspected in May, 1908. Internal and external conditions were found to be much the same at the latest examination as at the previous one. In 1908 the number of tubercules upon the interior surface was estimated to range from 3 to 70 per square foot, with very few sheets showing the larger number. There has since been a probable increase of from 10 to 20 per cent. The pittings beneath the tubercules have not grown materially deeper, and the tubercules have not so far as could be judged increased very greatly, either in size or in numbers. The average depth of the pittings now ranges from .075 in. to .090 in. The deepest one found was .110 in., which is no deeper than one found five years ago. The thickness of the steel plate is 5/16 or .312 in. It will thus be noted that this extreme pitting scarcely exceeds one-third of the thickness of the steel, and this pipe has now been in constant service for 17 years.

In the cast iron pipe the tubercules themselves have grown considerably in size. The extent of this growth is best indicated by the four outer rings, which can be clearly seen in Fig. 1, showing the under or contact side of tubercule. Photographs showing side view of sample tubercules from the cast iron pipe are shown in Fig. 2.

Samples of tubercules from the cast iron pipe which enters the High Hill reservoir gate house were collected and analyses of the soft central or middle portion thereof and of the tubercules as a whole were made. Results of these analyses are as follows:

Analyses of Tubercules.—Following are the analyses of the soft central portion of tubercules from the 48-in. cast iron pipe which enters High Hill reservoir gate house and of a whole tubercule:

	Central part of tubercule.	Whole tubercule.
Silica and insoluble matter....	3.24	1.78
Ferric oxide Fe_2O_3............	69.12	68.62
Alumina and lime Al_2O_3 and CaO	traces	traces
Sulphuric anhydride SO_3........	1.36	0.55
Water of hydration H_2O.......	26.28	28.16
Carbon dioxide, CO_2............	none	none

The interior of the steel pipe is now plentifully lined with blisters as well as tubercules. The blisters, however, far outnumber the tubercules. The number of blisters has somewhat increased within the past five years. In a way they help to protect the steel, for there is seldom any trace of rusting found upon the steel beneath them. Under the blisters the steel is generally sound and smooth and in most instances the mill scale, or coating of magnetic oxide of iron formed during the rolling of the plate, is intact. The water held by these blisters is of an alkaline nature. It is not as alkaline now as it was five years ago. Occasionally, under the blisters, rust is beginning to appear, but not yet sufficient to affect the surface of the steel. These blisters are of all sizes up to 2 and 3 ins. in diameter.

The pipe was also entered at the "Beaver Dam." This section was previously examined on May 22, 1908, and again on Oct. 22, 1909. The conditions were found to be generally similar to those at the time of the last examination, although it is quite certain that the number of blisters has been increasing while the tubercules have remained practically constant as to numbers. The deepest pitting found was 0.112 ins., which shows no increase over the extreme depth found five years ago. Here were found frequent small patches of spongilla, none of which were seen in the High Hill section. Later in the season these will be very much larger in all parts of the pipe.

The inside asphalt coating in the steel pipe has lost most of its original elasticity, but the facts that new blisters have formed and that old ones have increased in size seem to indicate that some elasticity still remains and that the coating is still of value as a protection to the steel. Walking through this pipe breaks up this coating, and there seems to be no question but that the less it is disturbed the better it is for the pipe.

It seems probable that the carrying capacity of this pipe has not changed much from that of the two previous examinations, but this is due in part to the low velocity of flow through it. In the various places where the outside coating was examined it was found to be very smooth and in excellent condition.

Thus far there have been very few reliable data upon which to base the probable life of a steel pipe of this character, though this life is dependent on the quality of the water carried. When this pipe was laid 17 years ago, it was expected to last at least 25 years. Previous to this installation the comparative merits of steel and cast iron pipe was the subject of much discussion by the water board, with their constructing engineers, Messrs. George S. Rice and George E. Evans, who convinced the board that the cost of the steel pipe would be about 28 per cent less than that of a cast iron pipe of same size, and that the steel pipe would have an assured life of at least 25 years. Thus the city could well afford to replace the steel pipe every 25 years rather than to pay the extra cost of installing the cast iron. That this has proved to be a very wise decision is now confirmed by the fact that in the opinion of those who have just examined this pipe, it is still in excellent condition. They find the interior conditions to be such that a long life is still in sight. It seems that the pitting of the steel upon the interior surface instead of advancing by penetrating the steel more deeply, has, to some extent at least, been active in spreading itself out over somewhat larger areas and to lesser depths. So with this 17 years' experience, it is now very easy to prophesy its reliability and usefulness for many years exceeding the 25 year limit.

Design Features of the New Rapid Sand Filtration Plant at Waco, Texas.

The city of Waco, Texas, formerly secured its water from wells driven in the bed of the Brazos River. As the consumption of water increased, it became necessary to increase the supply, and experience with the wells was not such as to encourage the idea that a sufficiently increased supply could be secured to meet the needs of the city.

The Brazos River flows through, or rather, immediately beside the city, and it was determined after a very thorough and careful investigation of the whole project to build an entirely new waterworks plant. In the spring of 1912 a bond issue of $400,000 was passed for this purpose and the Board of Commissioners selected as the consulting engineer, for the elaboration of their plans, Mr. N. Werenskiold of Dallas, Texas. His plans were drawn after a thorough study of the whole situation and contemplated the construction of a new pumping station, taking water from the river and filter plant to purify it, with clear water storage reservoir, all located some distance above the present plant, and on the side of the river on which the city is situated.

For the construction of the filter plant, in the fall of 1912, general specifications were drawn, giving the general outline of the size of plant required, the type of construction, etc., and the details of equipment and design were left by the city to the bidders to work out and submit with their bid, it being a general understanding that the city desired to put in a thoroughly first-class and up-to-date filtration plant. The design of the plant was described in a paper before the Southwestern Waterworks Association by Mr. F. B. Leopold, general manager of the Pittsburgh Filter Manufacturing Co. of Pittsburgh, Pa. The information here given is taken from Mr. Leopold's paper.

In general, the plant consists of coagulating basins in duplicate, each having capacity of about 600,000 gals., making a total capacity of about 1,200,000 gals., or 6 hours sedimentation for 5,000,000 gals. filter capacity.

The filter plant consists of four units of 1,225,000 gals. capacity each and the clear well storage in a covered rectangular concrete basin, with capacity of 1,000,000 gals. The plant is entirely of reinforced concrete construction, and is covered with a brick building with a tile roof.

There were some unusual conditions in the construction of this plant due to the site. The fact that the entire plant was built on a ledge of solid rock which had to be cut out from the surface the entire depth of the basins for the installation of the basin and filters, as well as the clear well, the top of the settling basin wall extending only about 18 ins. above the top of the rock surface and about 15 ft. into the rock. The large amount of rock to be taken out consumed a considerable time, being rather tedious work, and although the plant was contracted for in November, 1912, it was not completed until about the 1st of December, 1913, and only placed in actual operation in April, 1914, due to the fact that the pumping station and other work was not far enough advanced to operate.

This plant has a number of features that are incorporated in no other plant of its size in the country and are for that reason somewhat unique. The low service pumps are Allis-Chalmers direct connected, engine driven turbine pumps, and are located in the pit of the main pumping station. They take the water from the intake conduit leading from the Brazos River, from which point it is pumped to the inlet or mixing chamber, so-called, of the filtration plant. These pumps are in duplicate. On the steam supply line, leading to the engines, is an automatic chronometer valve that regulates the supply of steam to the engine, and is in turn regulated by the water level in the coagulating basin so that the pumpage is practically automatically taken care of—in other words, when the water rises to a certain elevation in the settling basin, the steam is automatically cut off from the engine and the pumps are stopped. As the water level drops, the steam is gradually admitted and the pumps are automatically started in operation again, so that there is little attention required between the filter plant and the pumping plant to maintain uniform operation.

View of the Rapid Sand Filtration Plant at Waco, Texas.

The water is delivered into the inlet or mixing chamber at the filtration plant, this chamber holding about 150,000 gals. of water or practically ¾ hour supply to the filters and is highly baffled with a series of up-and-down wooden baffles in order to insure a thorough mixture of the chemical solutions with the water previous to the entrance to basin. From this chamber are arranged inlets to each basin, so that they may be operated in parallel. There is also a by-pass connection from basin No. 1 to basin No. 2, enabling the operation of the basin in series—in other words, the water passing from the mixing chamber is taken through basin No. 1 and brought back to the inlet of basin No. 2, passing through basin No. 2, and then to the filters. There is also a by pass direct from the mixing chamber to the filters without going through either basin so that there is a possibility of operating with both settling basins out of commission, thus giving a very considerable flexibility in the operation of the plant.

The settling basins are about 115 ft. in length and 50 ft. in width, with a depth of 16 ft., and have a solid dividing wall between them, so that they can be operated either in parallel, in series or independently of each other. This gives an opportunity of giving almost any desired period of sedimentation after the addition of coagulant, within the limit of the ultimate capacity of both basins

After leaving the basin, the water is passe
through a conduit between the two rows o
filters and distributed to each individual filte
through a hydraulically operated gate valve
The head house, containing the coagulant solution plant, storage room and laboratory, i
built over a portion of the settling basin an
is a two-story structure, the first floor bein
used for the solution tanks, coagulating regulating apparatus and laboratory; the secon
floor for the mixing of chemicals and th
storage of the coagulants. This arrangemen
also means an extremely short coagulant solution line, thereby reducing to a minimum
troubles due to incrustation in coagulan
lines.

The plant is designed for the use of lim
and iron and hypo, but also can be used fo
alum if desired. It has, however, so far bee
used with lime and iron and the results hav
proven most excellent with a turbidity of a
great as 22,000 p. p. m.

The solution tanks consist of six tanks, o
a pair of tanks for each solution, with dissolving boxes for mixing located in the sec
ond floor. Each tank is equipped with a
electrically driven agitator, for agitating th
solution, and the building is equipped wit
a hydraulic elevator for raising the coagulan
from the first floor up to the storage room.

In the storage room there is a track wit
a traveler extending around the room an
over the top of the solution tanks, so th
the chemicals can be raised up to the travel
and carried in the required quantities over t
the dissolving tank with the least amount
labor.

On the first floor is located the chemic
regulating apparatus. This consists of t
Earl automatic controlling devices. These devices were illustrated in ENGINEERING A
CONTRACTING of Nov. 13, 1912. Their fun
tion, however, is to apply chemical soluti
automatically in proportion to the supply
water delivered to the coagulating basin. T
apparatus consists of what is known as t
master controller and the individual chem
cal controllers, one apparatus for each of t
chemicals provided.

The master controller is regulated by t
velocity of the flow of water through t
supply pipe to the coagulating basin, whic
through a pitot and pitometer tube builds
a head in the master controller correspondi
to the quantity of water passing through t
inlet pipe, and this head of course automa
ically changes with the quantity passing
the coagulating basin and is transferred fr
the master controller to the individual che
ical controllers, and in turn regulates prop
tionately the head of the chemical soluti
over the orifice feed and in this way gives
absolute proportional feed of the chemi
solutions to the quantity of water filter
thus securing the very highest possible deg
of accuracy and economy in the use of t
coagulants.

While this apparatus is somewhat elabor
and costly, the economy in operation fu
justifies their use, although in smaller pla
it is not generally customary to do so. T
same type of apparatus is in use in N
Orleans, Minneapolis, Grand Rapids, M
treal and other places, but only in one oth

plant of the size of the Waco plant or anywhere nearly approaching it, that is at Fargo, N. D.

By the use of an absolutely accurate automatic treatment there is of course the advantage of the highest economy in the use of chemicals as well as the knowledge that at all times the water is properly treated and not undertreated at one time and at another time overtreated, so that the uniformity of treatment is one depending upon the attention of the operator in plants that are not equipped with this type of apparatus.

In addition to the automatic control each of the solution tanks is equipped with a recording gage which gives a definite record of the solutions that are used each day, thus giving a check on the operation of the plant also. The solution apparatus being located directly over the inlet chamber, the supply line to the inlet chamber is extremely short and the usual troubles from this source are practically eliminated.

On this floor is located also the laboratory, which is complete in its equipment, and up to the standard found in most of the larger plants in the country.

The filter building proper consists of a one-story building extension from the head house and the filters are four in number, two located on each side of a central gallery. They are equipped with hydraulic valves, operating tables and recording loss of head gages, which give a record of the operation of the filters.

The operating tables consist of a marble top, built of gray Tennessee marble, on which are mounted the apparatus for controlling the hydraulic valves and the indicators showing the position of the valves; also the loss of head gage and a sampling device through which a sample of the filtered water is kept continually in circulation, so that the operator or visitor may see at any time just the condition of water from each filter and may secure his samples also for analysis from these points. These tables are duplicates of those used in the 60,000,000 gal. plant at Minneapolis and the 160,000,000 gal. plant at St. Louis, Mo., and are the very best type of construction. The design, construction and cost of the Minneapolis plant were very fully described in ENGINEERING AND CONTRACTING of June 11, June 25 and Nov. 5, 1913. The design of the St. Louis plant was described in ENGINEERING AND CONTRACTING of March 18, 1914.

The filter beds themselves are rectangular concrete units and are equipped with both air and water for washing purposes. Each bed is equipped with three wash troughs of a diamond shape, open at the top. The peculiar shape of this design is to prevent the loss of sand during the washing operation that occurs where a straight side trough is used. The water delivered into the filters from a conduit that is carried through between the two rows of filters and passes into an inlet chamber at one end and from there is distributed through the overflow troughs to the sand bed, passing down through the sand bed in the usual manner and is carried out through the strainer system consisting of central manifolds with 2-in. cast iron laterals spaced on 2-in. centers, extending from manifold to walls of filter chamber, and having fastened into them on 6-in. centers, brass strainers. From here it passes through the effluent into the gallery and there passes through the Earl type of automatic rate controller into the clear well. This rate controller provides against the filter operating to exceed its certain maximum rate, but within the maximum rate it automatically controls the filters to maintain a uniform level of water in the clear well; in other words, as the consumption decreases and the water level in the clear well rises, the filters are gradually closed off and the rate of flow decreased until when they arrive at a point where the maximum level of the water in the clear well is reached they are completely closed off without attention from the operator. If the service pump then operates at a rate requiring the full capacity of the filters to supply them, the water gradually receding the clear well, the filters are automatically and gradually opened to supply the increasing need until the limit of maximum flow is reached, which they are not allowed to exceed but which will be maintained until the consumption has so decreased again that the water level in the clear well gradually rises and again puts them under control. This is accomplished by a master controller which controls all of the individual rate controllers. This master controller is connected to the clear well and the difference in the level in the clear well is used to control the rate of filtration in the filter. The master controller operated by this head then in turn connects to each one of the individual rate controllers. There is also the advantage, if that is desired, that each individual filter rate controller can be disconnected from the master controller and operated independently as a fixed rate controller, or as an adjustable rate controller, in this way giving the greatest flexibility of control of any apparatus that has so far been devised for this purpose.

It will be seen, therefore, that this plant is practically as nearly automatic in its control from the time the water enters the settling basin until it is taken out of the clear water basin as it seems possible to devise, and it is the only plant that the writer knows of so thoroughly equipped with automatic devices. Many of the larger plants have some of these devices but I know of no plant that is completely equipped with them. All of the regulating and controlling devices were manufactured and supplied by the Pittsburgh Filter Manufacturing Co.

For washing, a steel tank is used instead of the usual wash pump, for several reasons, one being, of course, that under equal conditions the wash tank is preferable as it gives a uniform pressure that is almost impossible to secure from a pump; another one is that conditions here were hardly equal. The water could be supplied to the wash tank with the high duty pump, probably a little more cheaply than it could be pumped with the ordinary centrifugal pump direct, also the clear water well being located some little distance from the filter plant connections would have been rather inconvenient and complicated to make, to use the pump. The wash tank is supplied automatically from the main and when filled the supply is closed. As the washing operation is started and the water level in the tank begins to recede, the supply valve is automatically opened and remains open until the water level in the tank again reaches its maximum level when it is automatically cut off.

Southwestern Practice in the Purchase and Testing of Pump for Water Works Service.

Progress in water supply engineering has neither been accidental nor sudden but a scientific development based on mistakes, experience, collection of cost data, efficiency reports and tests, efforts to attain economy of operation and above all the application of scientific principles of hydraulic engineering to the needs and requirements of the various communities. As it is necessary to pump the entire water supply, except in rare instances, the selection of the type of pump best suited to local requirements is one of the water works engineer's first concerns. The present article describes the prevailing practice in the southwest in the purchase and testing of pumps, as given by Mr. Charles Schultz, City Engineer of Muskogee, Okla., in his paper before the third annual convention of the Southwestern Water Works Association.

Early in the development of modern machinery it became necessary on account of the different designs on the market, to test by the various methods in use at that time those offered for sale. The many claims made by the manufacturers for their particular pump as well as the different tests insisted upon by various engineers, resulted in untold confusion. This created so much dissatisfaction on account of the wide variation in results that in 1902 the American Society of Mechanical Engineers adopted a standard procedure which is now generally followed for conducting tests on all classes of pumps, engines and accessories.

In considering the efficiency of a pumping unit we speak of the duty it develops, which represents the ratio of the work done to the energy expended in doing it; this may be expressed in several ways, as per 100 lbs. of coal, 1,000,000 heat units, or 1,000 lbs. of dry steam, the second being the most desirable, yet the latter the most commonly used.

A brief survey of the pumps used in the ordinary pumping stations shows a wide variation of efficiency as follows:

Type of pump.	Duty in 1,000,000 ft. lbs. per 1,000 lbs. of dry steam.
Turbine or engine driven centrifugal...	70-90
Horizontal duplex direct acting compound	50-75
Horizontal or vertical duplex direct acting triple	75-110
Horizontal cross compound crank and fly wheel	110-150
Vertical triple expansion crank and fly wheel	140-205.5

This last and highest duty having been attained quite recently in the test of a new 20,000,000 gal. pump installed for the city of St. Louis, which is, I believe, the highest record attained to date.

In specifications as usually drawn a duty somewhere within the above limits is called for with or without a bonus or forfeiture clause as the purchaser may choose. The variation in duty being considerable, it is quite evident that the proper duty to specify for given conditions is often not easy to decide upon. In most instances sealed bids for pumps are asked for under the specifications, all bidders being permitted to submit alternate specifications and bids with the result that in many instances the item of price too largely controls the selection and final decision. It is at this period in the purchase of a pump that the experienced salesman appears to best advantage, for he can often set forth the good qualities of a poor pump so well that the recommendation of the engineer is lightly considered and the purchaser has acquired another piece of machinery not adapted to the work it will be called upon to perform.

In the selection of the best pump offered by bidders, the various factors to be considered, many of which are affected by local conditions, are as follows:

Cost of land.
Cost of foundations.
Cost of building.
Cost of pump.
Cost of boilers required to supply steam to engine.
Cost of fuel.
Cost of maintenance, repairs, lubricants and waste.
Cost of accessories.
Efficiency of unit.

Where land is valuable, as in our large cities, the first cost of a site together with the housing of the installation has a most important bearing upon the final cost. Another application of this same item is seen in the deep well, where centrifugal pumps occupying the same space or less are delivering double the quantity of water handled by a larger, but more efficient pump.

In 1912 the writer installed for the city of Muskogee, one 6,000,000 gal. low duty centrifugal pump with an expense of $100 for foundation, while the cost of foundation for a horizontal cross compound high duty pump of like capacity installed at the same time was $900.

The boiler and fuel required to furnish steam to the engine are very important factors, for we not only add to the fixed charges, but also to the operating cost by purchasing an engine of large steam consumption and low efficiency. This may be justifiable, however, when cheap fuel is available to furnish steam at a low cost, as is possible with natural gas, or oil as in Muskogee.

Without a doubt, the least considered of all the above items is that of maintenance and repair, for very little reliable information applicable to the varying conditions to be met in practice is available. The accessories often are an important feature, the writer having personal knowledge of a condenser being installed that cost two-thirds as much as the pump it served.

It is evident from the foregoing that the cost and efficiency are not the only items that should be considered in the selection of a pump. Give a clever salesman a moderate price and a high guarantee of duty and he will succeed in having his pump selected at the average letting.

Many pumps no doubt, have been purchased because of a high duty guarantee, the bidder trusting to some future supplemental agreement to aid him on the day of the acceptance test. This test is usually run two or three months after the pump has been in service, being conducted by a representative of each party to the contract.

It is at this test that the company furnishing the machinery has the distinct advantage for its representative is a mechanical engineer, who does testing exclusively and knows the refinements that count in the final results, in fact an expert in this particular branch of engineering.

The procedure to be followed in conducting the test is given in a general way in the Code of the American Society of Mechanical Engineers above mentioned, and will not be enumerated here in detail, it being sufficient for the object of this paper to say that the principal items to be determined are the weight of water pumped in a given time together with the amount of power consumed in doing same.

It is in the determination of these factors that experience and skill count most for the reasons as ably set forth in the Code in the following language: "No specific rule can be laid down regarding many of the preparations to be made for a test, so much depends upon the local conditions; and the matter is one which must be left mainly to the good sense, tact, judgment and ingenuity of the party undertaking it."

Too few indeed are the pumping stations equipped with Venturi meters for measuring accurately the quantity of water pnmped, with the result that in many cases where basins are not available to pump from the pitometer, wier or other appliances are made use of. As all these require skill along particular lines, they are not in general use, resort being had to the leakage test to determine the valve slippage.

Given the number of revolutions and plunger displacement the quantity of water pumped during a test is determined by making due allowance for the slippage, which is too often only estimated.

The determination of the amount of energy used in a certain time is obtained by weighing the amount of condensed steam, making due allowance for moisture and pressure of the live steam, pressure in condenser, air pressure, temperature of air and several other variables that affect the final results.

The above observation although touching upon many points elementary in their character are intended to show to those not versed in this line of engineering, that the proper testing of a pump calls for a man possessing judgment, skill and experience in this particular line of work.

Since the cost of the pumping station is but a small part of most water works systems, it has become the custom in a majority of instances to have the civil engineer to design it along with the rest of the plant, the result being that the best arrangement possible is not always secured.

The particular point which the writer desires to impress and with all due deference and respect to the abilities of the average designing engineer, is that in imposing this duty upon him the client does not obtain the best possible results, for in most instances the engineer has not had the advantage of the special training and experience necessary to properly equip him for this particular line of duty; placing this burden upon him is unfair and too often the hidden fear of unjust criticism, or charge of lack of knowledge deter him from asking for and insisting upon expert advice of an acknowledged specialist or authority in this particular line of engineering work. False ideas of economy too often control city officials in the employment of expert service for unusual or particular problems, as well as blinding them to what they at the time consider extravagant e
penditures for testing apparatus.

Low insurance rates securing a saving
every property owner are inevitably the res
of an efficient water service, this end bei
striven for alike by the insurance compan
as well as the insured.

To provide for a reliable common source
expert advice and service at a nominal c
as well as securing almost free use of pro
testing apparatus, the writer ventures to ma
a suggestion which he trusts will not seen
radical departure, nor in the practical appli
tion of which will the duties and prerogati
of the consulting municipal engineer be
vaded, or encroached upon.

I would respectfully suggest that the sco
of the powers of the State Insurance Co
missioner be enlarged so as to embrace
employment of an expert hydraulic engin
and the purchase of proper equipment for
usual hydraulic tests, the service of the en
neer and testing equipment to be furnish
free or at nominal cost of municipalities
questing the same, or if found desirable
might be made obligatory upon municipalit
to have their plans for water works syste
approved by their department.

To recapitulate, we would have a cent
source of authority and information wh
reliable expert advice could be secured at l
cost, thereby securing for the cities the high
efficiency at the lowest cost, effecting a sav
to property owners in reduced taxes and
duced insurance rates. The municipalities,
surance commissioners and underwrit
would all be working and co-operating w
the same comon purpose in view; namely,
lowering of the fire risk. The honest ma
facturer would no doubt welcome such an
rangement for it would secure proper recog
tion of meritorious equipment.

The state of Wisconsin, through the
tension division of the university, is mak
preliminary reports for water and sewer
systems, the plans and details of which
later carried out by practicing engineers w
very satisfactory results to all parties c
cerned.

General

Costs Reported from a Number of American Tunnels.

A compilation of tunnel costs from data personally collected is given in Bureau of Mines Bulletin 57 by D. W. Brunton and J. A. Davis. Though similar figures for some of the same tunnels have been previously published, the tabulations are deserving of publication even at the expense of some repetition.

There are set forth in the following pages as complete and accurate data as could be obtained, showing the cost of various phases of tunnel work at a number of different tunnels. Although the writers have not had the advantage of auditing the books from which these figures were taken and hence can not vouch personally for the absolute accuracy of the figures, the data were in all cases procured from persons in charge or who were in a position to know what the work actually cost. Accompanying the figures is a brief list of the more important features of the tunnel, without which it is impossible to make even an approximate comparison between any two pieces of tunnel work.

GUNNISON TUNNEL.

Important Details.—Location: Montrose, Colo. Purpose: Irrigation and reclamation. Shape of cross section: Horseshoe. Size: 10 ft. wide at the bottom, 10 ft. 6 ins. wide at the spring line, 10 ft. high at the spring line, 12 ft. 4 ins. high at the center of the arch. Length: 30,645 ft. Character of rock penetrated: Chiefly metamorphosed granite with some water-bearing clay and gravel, some hard black shale, and a zone of faulted and broken rock. Type of power: Steam. Ventilator: Pressure blower. Size of ventilating pipe: 17 ins. Drills: At first, pneumatic hammer, 4 drills in the heading; afterwards, pneumatic, piston, 4 drills in the heading. Mounting of drills: Horizontal bar for the hammer drills, vertical columns for the piston drills. Number of holes per round: 20 to 24 in the heading (approximately one-half of the tunnel). Average depth of round: 6 to 7 ft. Number of drillers and helpers per shift: 4 drillers and 2 helpers. Number of drill shifts per day: 3. Explosive: 60 per cent gelatin dynamite, with some 40 per cent. Number of muckers per shift: 5 to 8. Number of mucking shifts per day: 3. Type of haulage: Electric. Wages: Drillers, $3.50 and $4; helpers, $3 and $3.50; muckers, $2.50 and $3, blacksmiths, $3.50 and $4; motormen, $3; brakemen, $2.50 and $3; power engineers, $4. Maximum progress in any calendar month: 449 ft. Average monthly progress: 250 ft., approximately.

Cost of driving

	Cost per foot of tunnel
10,019 feet driven by underc[illegible] he[illegible]d[illegible] and subsequent enlargement	$87 23
20,626 feet driven by top heading and bench	62 18
Average cost of excavation of entire tunnel	70 36

These costs include all labor, all materials, all repairs, all power depreciation figured as 100 per cent on all equipment, with a proportionate charge for general (supervisory) and miscellaneous expenses of the entire reclamation project.

LARAMIE-POUDRE TUNNEL

Important Details.—Location: Home, Colo. Purpose: Irrigation. Cross section: Rectangular. Size: 9½ ft. wide by 7½ ft. high. Length: 11,306 ft. Character of rock penetrated: Close-grained red and gray granite. Type of power. Hydr
at the east end, electric at the west. Ventila
Pressure blower. Size of ventilating pipe:
and 15 ins. Drills: 3 pneumatic ham
Mounting of drills: Horizontal bar. Numbe
holes per round. 21 to 23. Average dept
round 10 ft. at first, 7 to 8 ft. later. Numb
drillers and helpers per shift: 3 drillers, 2 h
ers. Number of drill shifts per day: 3.
plosive. 60 per cent gelatin dynamite, with s
100 per cent in the cut holes. Number of m
ers per shift: 6. Number of mucking shifts
day: 3. Type of haulage: Mules. Wages. D
ers, $4.50, helpers, $4, muckers, $3.50; bl
smiths, $5; drivers, $4 50, dumpmen, $
Maximum progress in any calendar month
ft., March, 1911 Average monthly progress
ft. (for the 16 months when complete p
operated). Special feature: Inaccessibility,
tunnel was located about 60 miles from
nearest railroad siding, and the roads
mountainous and very steep in places

Cost of driving tunnel 11,306 ft.

	Cos fo tur
Superintendents and foremen	
Drilling	
Mucking and loading	
Tramming and dumping	
Track and pipe	
Power house	
Blacksmithing	
Repairs	
Bonus to workmen	
Maintenance of camps, buildings, and fuel	
Machinery repairs	
Air drills and parts	
Picks, shovels and steel	
Explosives	
Lamps and candles	
Oil and waste	
Blacksmith supplies	
Liability insurance	

ffice supplies, telephone and bookkeeping	.86
	$29.81
ermanent equipment (less approximately 10 per cent salvage)	9.73
	$39.54

The permanent equipment included power lant, camp buildings and furnishings, pipes, ails, etc.

LOS ANGELES AQUEDUCT.

Little Lake Division, Tunnels 1 to 10A.

Important Details.—Location: Inyo County, al. Purpose: Water supply, power, and irrigaon. Cross section: See Fig. 1. Size: See Fig. 1. ype of power: Electric power purchased at a ominal cost per kilowatt-hour from a hydraulic lant constructed and owned by the aqueduct. entilators: Pressure blowers. Size of ventilatg pipe: 12 ins. Drills: Pneumatic hammer, sually 2 in each heading. Mounting of drills: orizontal bar. Number of holes per round: sually 14 to 16. Average depth of round: 6 to ft. Number of drillers and helpers per shift: drillers and 2 helpers. Number of drill shifts er day: Usually 1, but sometimes 2. Explosive: per cent gelatin dynamite, with some 20 per ent and some 60 per cent; ammonia dynamite lso tried. Number of muckers per shift: sually 5. Number of mucking shifts per day: sually 1, but 2 when 2 drill shifts were emloyed. Type of haulage: Tunnels 1 to 3-N, ul[illegible] tunnels 3-S, to 10A-N, electric; tunnel 0A-S, mules. Wages: Drillers and helpers, $3; uckers, $2.50; blacksmiths, $4; helpers, $2.50; otormen, $2.75; dumpmen, $2.50.

OST OF DRIVING TUNNEL 1B-S FOR 1,341 FT.

Driven through medium-hard granite at an average speed of 225 ft. per month.*]

	Cost per foot of tunnel.
xcavation	$9.15
ngineering	.18
dit proportion	.28
ermanent equipment (estimated)	2.35
imbering (857 ft.)	1.02
	$12.98

---

*The average speed given is computed on the asis of one heading per month.

In this tunnel, as in all of the tunnels of this ivision and of the Grapevine division, the cost f excavation includes the wages of shift foreen, drillers, helpers, muckers, motormen or ule drivers, dumpmen, blacksmiths and helprs, machinists, electricians (part), and power ngineers; also the cost of powder, fuse, caps, andles, light globes, machine oil, blacksmith upplies and fuel, and machinists' supplies, and e cost of power and of repairs for power, aulage, compressor, and ventilating machinery.

"Engineering" includes the cost of giving line nd grade, etc.

"Adit proportion" is a proportionate charge er foot of tunnel to defray the cost of an adit om the surface to the tunnel line.

"Permanent-equipment" costs were not segreated for each tunnel, but were compiled for e whole division, so the charge represents a roportionate charge per foot for the entire ivision cost, without salvage, of trolley and ght lines, including freight and cost of installation; pressure air lines with freight and stallation; ventilating lines with freight and stallation; water lines with freight and installation; mine locomotives and cars, picks, ovels, drills and drill sharpeners, with repairs r the last four items.

OST OF DRIVING TUNNEL 2, LENGTH 1,739 FT.

Driven through medium-hard but very wet granite at an average speed of 170 ft. per month.]

	Cost per foot of tunnel.
xcavation	$8.81
ngineering	.19
dit proportion	.34
ermanent equipment	2.35
imbering (1,590 ft.)	3.28
	$14.97

OST OF DRIVING TUNNEL 2A, LENGTH 1,322 FT.

Driven through medium-hard granite at an average speed of 150 ft. per month.]

	Cost per foot of tunnel.
xcavation	$8.05
ngineering	.16
dit proportion	.34
Permanent equipment	2.35
Timbering (1,322 ft.)	2.51
	$13.41

COST OF DRIVING TUNNEL 3-N FOR 1,148 FT.

[Driven through medium-hard granite at an average speed of 150 ft. per month.]

	Cost per foot of tunnel.
Excavation	$10.10
Engineering	.23
Adit proportion	.51
Permanent equipment	2.35
Timbering (956 ft.)	2.44
	$15.63

COST OF DRIVING TUNNEL 3-S FOR 1,358 FT.

[Driven through granite of variable hardness, and containing pockets of carbon-dioxide gas, at an average speed of 155 ft. per month.]

	Cost per foot of tunnel.
Excavation	$12.38
Engineering	.28
Adit proportion	.16
Permanent equipment	2.35
Timbering (1,244 ft.)	3.28
	$18.45

COST OF DRIVING TUNNEL 3 (3-N AND 3-S), COMPLETE, 4,044 FT.

[Driven through decomposed granite of medium hardness, dissected by slips and talcose planes requiring timber where ground was wet, and also containing pockets of carbon-dioxide gas, making work difficult and requiring extra provisions for ventilation. Average speed, 140 ft. per month.]

	Cost per foot of tunnel.
Excavation	$12.67
Engineering	.24
Adit proportion	.35
Permanent equipment	2.35
Timbering (3,570 ft.)	2.71
	$18.32

COST OF DRIVING TUNNEL 4, LENGTH 2,033 FT.

[Driven through medium hard to hard granite at an average speed of 145 ft. per month.]

	Cost per foot of tunnel.
Excavation	$12.00
Engineering	.24
Adit proportion	.16
Permanent equipment	2.35
Timbering (1,705 ft.)	2.16
	$17.01

COST OF DRIVING TUNNEL 5, LENGTH 1,178 FT.

[Driven through medium hard to very hard granite at an average speed of 120 ft. per month.]

	Cost per foot of tunnel.
Excavation	$11.10
Engineering	.21
Adit proportion	.08
Permanent equipment	2.35
Timbering (916 ft.)	1.83
	$15.57

COST OF DRIVING TUNNEL 7, LENGTH 3,596 FT.

[Driven through biotite granite of variable hardness at an average speed of 140 ft. per month.]

	Cost per foot of tunnel.
Excavation	$13.55
Engineering	.27
Adit proportion	.13
Permanent equipment	2.35
Timbering (2,609 ft.)	3.60
	$19.90

COST OF DRIVING TUNNEL 8-S FOR 1,334 FT.

[Driven through medium hard to hard granite at an average speed of 135 ft. per month.]

	Cost per foot of tunnel.
Excavation	$12.82
Engineering	.19
Adit proportion	.18
Permanent equipment	2.35
Timbering (126 ft.)	.39
	$15.93

COST OF DRIVING TUNNEL 9 FOR 3,506 FT.

[Driven through medium hard to hard granite at an average speed of 195 ft. per month.]

	Cost per foot of tunnel.
Excavation	$12.19
Engineering	.18
Adit proportion	.07
Permanent equipment	2.35
Timbering (305 ft.)	.29
	$15.08

COST OF DRIVING TUNNEL 10 FOR 5,657 FT.

[Driven through medium hard to hard granite at an average speed of 200 ft. per month.]

	Cost per foot of tunnel.
Excavation	$13.50
Engineering	.19
Permanent equipment	2.35
Timbering (194 ft.)	.11
	$16.15

COST OF DRIVING TUNNEL 10A-N FOR 1,496 FT.

[Driven through medium hard to hard granite at an average speed of 165 ft. per month.]

	Cost per foot of tunnel.
Excavation	$13.02
Engineering	.13
Permanent equipment	2.35
Timbering (24 ft.)	.78
	$16.28

COST OF DRIVING TUNNEL 10A-S FOR 2,200 FT.

[Driven through medium hard to hard granite at at average speed of 200 ft. per month.]

	Cost per foot of tunnel.
Excavation	$12.37
Engineering	.20
Permanent equipment	2.35
Timbering (215 ft.)	1.15
	$16.07

GRAPE VINE DIVISION, TUNNELS 12 TO 17B.

Important Details.—Location: Kern County, Cal. Purpose: Water supply, power, and irrigation. Cross section: See Fig. 1. Size: See Fig. 1. Type of power: Electric power purchased from aqueduct plant. Ventilators: Pressure blowers. Size of ventilation pipe: 12 ins. Drills: Pneumatic hammer, usually 2 in each heading. Mounting of drills: Horizontal bar. Number of holes per round: Usually 18 to 20. Average depth of round: 6 to 8 ft. Number of drillers and helpers per shift: 2 drillers and 2 helpers. Number of drill shifts per day: Usually 2. Explosive: 40 per cent ammonia dynamite, but 60 per cent and 75 per cent gelatin dynamite were employed in hard ground. Number of muckers per shift: 4 or 5. Number of mucking shifts per day: Usually 2. Type of haulage: Electric after the first 400 to 500 ft. Wages: Drillers and helpers, $3; muckers, $2.50; blacksmiths, $4; helpers, $2.50; motormen, $2.75; dumpmen, $2.50

COST OF DRIVING TUNNEL 12, LENGTH 4,900 FT.

[Driven through hard granite at an average speed of 185 ft. per month.]

	Cost per foot of tunnel.
Excavation*	$22.10
Engineering*	.32
Permanent equipment*	2.35
Timbering (90 ft.)	.08
	$24.75

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*These items include the same costs as for the Little Lake division.

COST OF DRIVING TUNNEL 13 FOR 1,525 FT.

[Driven through hard granite at an average speed of 130 ft. per month.]

	Cost per foot of tunnel.
Excavation	$20.60
Engineering	.10
Permanent equipment	2.25
Adit proportion	.37
	$23.32

COST OF DRIVING TUNNEL 14, LENGTH 859 FT.

	Cost per foot of tunnel.
Excavation	$22.70
Engineering	.13
Permanent equipment	2.25
Adit proportion	.72
Timbering (22 ft.)	.16
	$25.96

COST OF DRIVING TUNNEL 15, LENGTH 895 FT.

	Cost per foot of tunnel.
Excavation	$23.28
Engineering	.11
Permanent equipment	2.25
Adit proportion	2.42
	$28.06

COST OF DRIVING TUNNEL 16, LENGTH 2,723 FT.

[Driven through hard granite at an average speed of 145 feet per month.]

	Cost per foot of tunnel.
Excavation	$20.07
Engineering	.17
Permanent equipment	2.25
Adit proportion	.55
Timbering (18 ft.)	.04
	$23.08

COST OF DRIVING TUNNEL 17, LENGTH 3,024 FT.

	Cost per foot of tunnel.
Excavation	$20.47
Engineering	.21
Permanent equipment	2.25
Timbering (142 ft.)	.22
	$23.15

COST OF DRIVING TUNNEL 17½ FOR 1,345 FT.

[Driven through medium hard to hard granite at an average speed of 225 feet per month.]

	Cost per foot of tunnel.
Excavation	$19.56
Engineering	.31
Permanent equipment	2.25
	$22.12

COST OF DRIVING TUNNEL 17A FOR 3,275 FT.

	Cost per foot of tunnel.
Excavation	$18.70
Engineering	.17
Permanent equipment	2.25
Timbering (441 ft.)	1.18
	$22.30

COST OF DRIVING TUNNEL 17B FOR 4,915 FT.

	Cost per foot of tunnel.
Excavation	$21.09
Engineering	.21
Permanent equipment	2.25
Timbering (163 ft.)	1.90
	$25.45

ELIZABETH DIVISION, ELIZABETH LAKE TUNNEL.

Important Details.—Location: Los Angeles County, Cal. Purpose: Water supply, power and irrigation. Cross section: Rectangular, with arched roof. Size: 12 by 12 ft. Length: 26,870 ft. Type of power: Electric power purchased from aqueduct plant. Ventilator: Pressure blower. Size of ventilating pipe: 18 ins. Drills: Pneumatic hammer, 3 in the south heading and 2 in the north. Mounting of drills: Horizontal bar. Number of holes per round: 25 in the south heading, 16 in the north heading. Average depth of round: 8 to 10 ft. Number of drillers and helpers per shift: 2 drillers and 2 helpers at the north end, 3 drillers and 3 helpers at the south end. Number of drill shifts per day: 3. Explosive: 40 per cent and 60 per cent gelatin dynamite. Number of muckers per shift: 6. Number of mucking shifts per day: 3. Type of haulage: Electric. Wages: Drillers and helpers, $3; muckers, $2.50; blacksmiths, $4; helpers, $2.50; motormen, $2.75; dumpmen, $2.50. Maximum progress in any calendar month: 604 ft., April, 1910. Average monthly progress per heading: 350 ft. per month.

COST OF DRIVING THE NORTH HEADING, ELIZABETH LAKE TUNNEL.

[Driven through altered granite, requiring much timbering, 13,370 ft.]

	Cost per foot of tunnel.
Drilling and blasting	$11.25
Mucking and tramming	11.70
Engineering and superintendence	1.27
Drainage	.45
Ventilation	.22
Light and power	5.55
Timbering (13,031 ft.)	8.48
Cost of auxiliary shaft	.93
Permanent equipment (full charge, no salvage; estimated)	3.70
	$43.55

COST OF DRIVING THE SOUTH HEADING, ELIZABETH LAKE TUNNEL.

[Driven through medium hard to hard granite, requiring but little timbering, 13,500 ft.]

	Cost per foot of tunnel.
Drilling and blasting	$14.65
Mucking and tramming	11.10
Engineering and superintendence	.86
Drainage	.17
Ventilation	.41
Light and power	4.93
Permanent equipment (without salvage; estimated)	3.70
Timbering (3,424 ft.)	2.19
	$38.01

LUCANIA TUNNEL.

Important Details.—Location: Idaho Springs, Colo. Purpose: Mine development and transportation. Cross section: Square. Size: 8 by 8 ft. Length: 12,000 ft. projected; 6,375 ft. driven December 1, 1911. Character of rock penetrated: Hard granite. Type of power: Purchased electric Current. Ventilator: Pressure blower. Size of ventilating pipe: 18 and 19 ins. Drills: Pneumatic hammer, 3 in the heading. Mounting of drills: Vertical columns. Number of holes per round: 25. Average depth of round: 8 to 9 ft. Number of drillers and helpers per shift: 3 drillers and 2 helpers. Number of drilling shifts per day: 1. Explosive: 50 per cent gelatin dynamite. Number of muckers per shift: 3. Number of mucking shifts per day: 1. Type of haulage: Horses. Wages: Head driller, $5; drillers, $4; nipper, $3.50; boss mucker, $5; muckers, $4; drivers, $4; power engineers, $4; blacksmith, $5. Maximum progress in any calendar month: 263 ft., September, 1911. Average monthly progress: 125 ft. per month for the first 4,800 ft., 240 ft. per month for the last 1,575 ft.

AVERAGE COST OF DRIVING FIRST 4,800 FT.

	Cost per foot of tunnel.
Labor	$8.86
Powder	7.86
Fuse and caps	.17
Candles and oil	.21
Horse feed and shoeing	.18
Power	1.64
Repairs	.14
Tunnel equipment	2.75
Surface plant	1.25
	$23.06

"Tunnel equipment" includes the cost of materials and installation of the pressure air line, the ventilating line, rails, ties and fittings, and the drainage ditch. "Surface plant" includes buildings, compressor blower, transformers, motors and drill sharpener.

Cost of driving next 1,575 ft.: The contractor received $21.50 per foot to cover the cost of labor, powder, fuse, caps, candles, oil, horse feed and shoeing, power and repairs, and the installation of the tunnel equipment.

MARSHALL-RUSSELL TUNNEL.

Important Details.—Location: Empire, Colo. Purpose: Mine drainage, development and transportation. Cross section: Rectangular. Size: 8 ft. wide by 9 ft. high. Length: 11,000 ft. projected: 6,700 ft. driven January 1, 1913. Character of rock penetrated: Granite and gneiss. Type of power: Purchased electric current, also a small auxiliary hydraulic plant. Ventilator: Fan. Size of ventilating pipe: 12 and 13 ins. Drills: 2, pneumatic hammer. Mounting of drills: Vertical columns. Number of holes per round: 18 to 20. Average depth of round: 9 to 10 ft. Number of drillers and helpers per shift: 2 drillers and 2 helpers. Number of drill shifts per day: 1. Explosive: 40 per cent gelatin dynamite; with some, 80 per cent. Number of muckers per shift: 4. Number of mucking shifts per day: 1. Type of haulage: Horses. Wages: Drillers, $4; helpers, $3; blacksmith, $4; helpers, $3; muckers, $3.25; trammers, $3.75; dumpmen, $3.25; power engineer, $3.50; shooters, $3.25. Maximum progress for any calendar month: 187 ft., June, 1912. Average monthly progress: 125 ft.

COST OF DRIVING TUNNEL 6,700 FT.

	Cost per foot of tunnel.
Labor	$9.17
Powder, fuse, caps and blacksmith cost	3.55
Drills, steel and repairs (less 30 per cent salvage)	1.34
Power	1.41
Permanent equipment and general expense (less 20 per cent salvage on permanent equipment)	3.41
	$18.88

MISSION TUNNEL.

Important Details.—Location: Santa Barbara, Cal. Purpose: Water supply. Cross section: Trapezoid. Size: 6 ft. wide at the base, 4½ ft. wide at the top, 7 ft. high. Length: 19,560 ft. Character of rock penetrated: Shale, slate, and hard sandstone. Ventilator: Pressure blower. Size of ventilating pipe: 10 ins. Drills: 1 pneumatic hammer. Mounting of drills: Horizon bar. Number of holes per round: 12 to Average depth of round: 7 to 8 ft. Number drillers and helpers per shift: 1. Number drilling shifts per day: 3. Explosive: 40 per ce and 60 per cent gelatin dynamite. Number muckers per shift: 4. Number of mucking shi per day: 3. Type of haulage: Electric. Wage Drillers, $3.50; helpers, $3; muckers, $2. blacksmiths, $4; helpers, $3; motormen, $2. dumpmen, $2.50; power engineers, $2.75. Max mum progress in any calendar month: 414 f February, 1911. Average monthly progres 210 ft.

COST OF DRIVING THE SOUTH PORTA MISSION TUNNEL, MAY, 1909, TO SEPTEM BER, 1911 5,515 FT.

	Cost p foot tunne
Administration	$1.
Labor	9.
Power	2
Explosives	1.
Timbering (563 ft.)	.
Track and pipe	1.
Miscellaneous supplies	2.
Drill parts (including steel)	1.
Bonus	.
	$19.

"Administration" includes superintendenc office supplies, and general charges. "Misce laneous supplies" includes candles, light globe shovels, picks, blacksmiths' supplies and fu and machinists' supplies.

NEWHOUSE TUNNEL.

Important Details.—Location: Idaho Spring Colo. Purpose: Drainage and transportatio Cross section: Square. Size: 8 by 8 ft. Lengt 22,000 ft. Character of rock penetrated: Idal Spring gneiss. Type of power: Purchased ele tric current. Ventilator Pressure blower. Si of ventilating pipe: 18 ins. Drills: Pneumat hammer. Mounting of drills: Vertical colum Number of holes per round: 14 to 22. Numb of drill shifts per day: 1 and 2. Explosive: per cent gelatin dynamite, with some 100 p cent in the cut holes. Number of muckers p shift: 3. Number of mucking shifts per da 1 and 2. Type of haulage: Electric. Wage Drillers, $4 to $4.50; helpers, $3.25 to $4; muc ers, $3.50; motormen, $3.50; dumpmen, blacksmiths, $3.50 to $4.50; helpers, $3.

COST OF DRIVING THE NEWHOUSE TU NEL.

	Jan. to Aug., 1909, 1,633 ft.	Sept. to Dec., 1909, 1,098 ft.	Apr. to Aug.
Labor	$6.72	$6.98	$11
Explosives	4.15	3.72	4
Fuse and caps	.39	.36	
Transportation of materials broken	1.49	1.47	
Power	1.99	2.16	
Blacksmithing	1.57	2.61	
Use of drills, repairs and steel	1.50	2.74	
Equipment, ties, rails, pipe, etc.	1.71	1.78	
Sundries	.79	.80	1
	$20.31	$22.42	$3

RAWLEY TUNNEL.

Important Details.—Location: Bonanza, C Purpose: Mine drainage and development. Cr section: Trapezoidal. Size: 8 ft. wide at base, 7 ft. wide at the top, 7 ft. high. Leng 6,225 ft. Character of rock penetrated: Tou hard andesite. Type of power: Steam w wood for fuel. Ventilator: Pressure blow Size of ventilating pipe: 12 and 13 ins. Dri 2, pneumatic hammer. Mounting of drills: Ho zontal bar. Number of holes per round: 23 25. Average depth of round: 8 to 9 ft. at first to 6 ft. later. Number of drillers and help per shift: 2 drillers and 2 helpers. Number drill shifts per day: 2 at first, 3 later. Explosi 40 per cent and 60 per cent gelatin dynamite the proportion of about 2 to 1). Number muckers per shift: 4. Number of mucking sh per day: 2 and 3. Type of haulage: Horses mules. Wages: Drillers, $4.50, helpers, $3 muckers, $3.50; blacksmiths, $4.50; driv $3.50; power engineers, $4. Maximum progr in any calendar month: 585 ft., July, 1912. Av age monthly progress: Approximately 350 ft.

OST OF DRIVING THE TUNNEL 6,235 FT.*

	Cost per foot of tunnel.
rilling and firing	$5.25
ucking	2.16
ramming	1.13
rack and pipe	.44
iscellaneous underground expenses	1.41
ower plant	2.50
lacksmithing	.73
iscellaneous surface work	.83
eneral expenses	1.98
ermanent plant	3.24
imbering (1,618 ft.)	1.18
oarding house, debit balance	.04
	$20.98
redit by salvage on permanent plant	1.11
	$19.87

"Drilling and firing" includes labor, powder, se, caps, supplies, and repairs. "Mucking," Tramming," and "Track and pipe" include bor and supplies. "Miscellaneous underound expenses" include wages of foremen, iderground telephone, etc. "Power plant" inades labor, supplies, and fuel. "Blacksmithg" and "Miscellaneous surface work" include bor and supplies. "General expenses" include laries, office supplies, telephone, etc. "Peranent plant" includes machinery and buildgs, with labor of installation, steel rails, rmanent supplies, and repairs. "Timhering" cludes labor and supplies. The salvage of e permanent plant is approximately 50 per nt on salable articles, such as machinery, ils, cars, etc.

ROOSEVELT TUNNEL.

Important Details.—Location: Cripple Creek, ılo. Purpose: Mine drainage. Cross section: ectangular, with large ditch at the side. Size: ft. wide by 6 ft. high. Length: 15,700 ft. aracter of rock penetrated: Pikes Peak gran-, chiefly. Type of power: Purchased electric rrent. Ventilator: Purchased electric current. entilator: Pressure blower. Size of ventilating pe: 16 and 17 ins. Drills: 3, pneumatic hamer. Mounting of drills: Horizontal bar. Numr of holes per round: 24, usually. Average pth of round: 6 to 7 ft. Number of drillers d helpers per shift: 3 drillers, 2 helpers. umber of drill shifts per day: 3. Explosive: 40 r cent, 60 per cent, and some 100 per cent latin dynamite. Number of muckers per ift: 4, usually. Number of mucking shifts per y: 3. Type of haulage: Horses and mules. ages: Drillers, $5; helpers, $4; muckers, $3.50; wer engineer, $4; blacksmith, $5; helper, $3.50; impman, $3.50; drivers, inside, $5; outside, $4. aximum progress in any calendar month: 435 , portal heading, January, 1909. Average onthly progress: Portal heading, 300 ft.; shaft adings, 270 ft.; all headings, 285 ft.

COST OF DRIVING TUNNEL

otal cost of portal work	$111,980.06
ontractor's percentage	11,404.88
ost of shaft heading	262,126.55
Total cost of tunnel	$386,421.49
umber of feet driven	14,167
verage cost per foot	$27.27

ST OF DRIVING THE PORTAL HEADING.

	Feet	Cost. per foot.
08—		
February and March	511	$22.690
April	262	30.970
May	268	26.769
June	187	35.010
July	203	29.600
August	300	21.760
September	351	19.600
October	287	23.000
November	360	21.120
December	334	18.350
09—		
January	435	16.410
February	290	22.206
March	310	21.745
April	316	21.266
May	402	18.762
June (8 days)	62	40.606

COST OF DRIVING SHAFT HEADINGS.

	Feet	Cost. per foot.
08—		
October (2 headings)	49	$105.52
November (2 headings)	111	41.38
December (2 headings)	177	40.11
09		
January (2 headings)	261	21.06
February (2 headings)	601	23.70
March (2 headings)	639	26.256
April (2 headings)	670	25.02
May (2 headings)	552	28.34
June (2 headings)	498	27.375
July (1 heading)	319	32.871
August (1 heading)	410	27.747
September (1 heading)	355	32.40
October (1 heading)	380	28.178
November (1 heading)	298	34.20
December (1 heading)	251	35.153
1910—		
January (1 heading)	282	28.82
February (1 heading)	259	30.636
March (1 heading)	344	27.62
April (1 heading)	376	25.313
May (1 heading)	393	24.856
June (1 heading)	373	26.616
July (1 heading)	350	25.247
August (1 heading)	372	25.029
September (1 heading)	342	28.45
October (1 heading)	372	27.361
November (1 heading)	192	27.786

TYPICAL DISTRIBUTION OF EXPENSES, PORTAL HEADING, JULY, 1908, 203 FT.

	Cost per foot of tunnel.
Machinery and repairs	$0.61
Air drills and parts	.99
Picks, shovels and steel	1.90
Ditch men	1.09
Explosives	6.90
Candles	.36
Oil and waste	.09
Electric power	2.06
Blacksmith supplies	.09
General expense	.16
Liability insurance	.17
Lumber, ties and wedges	.01
Horses and feed	.01
Compressor men	1.79
Drillers and helpers	4.21
Blacksmiths and helpers	3.43
Muckers and drivers	4.11
Foremen	1.50
Bookkeeper	.12
	$29.60

TYPICAL DISTRIBUTION OF EXPENSES, SHAFT HEADING, FEBRUARY, 1910, 259 FT.

	Cost per foot of tunnel.
Maintenance of buildings, tents, etc.	$0.096
Machinery and repairs	1.158
Air drills and parts	1.930
Shovels, picks and steel	1.930
Pipe and fittings	.193
Ditch men	1.480
Explosives	5.032
Lamps and candles	.217
Oil and waste	.252
Electric power	2.440
Blacksmith supplies	.150
Liability insurance	.213
General expense	.342
Lumber, ties and wedges	.119
Horses and feed	.324
Machine men and helpers	4.050
Muckers	3.065
Blacksmiths and helpers	1.362
Engineers	1.300
Pipe and track men	.675
Drivers and dump men	2.355
Foremen	1.752
Mine telephone	.008
Bookkeeper	.193
	$30.636

STILWELL TUNNEL.

Important Details.—Location: Telluride, Colo. Purpose: Mine drainage and development. Cross section: Square, with ditch at side. Size: 7 by 7 ft. Length: 2,950 ft. Character of rock penetrated: Conglomerate and andesite. Type of power: Purchased electric current. Ventilator: Fan. Size of ventilating pipe: 10 ins. Drills: Started with electric drills, finished with pneumatic piston drills, using 2 in the heading. Mounting of drills Vertical columns. Number of holes per round: 16. Average depth of round: 6 to 6½ ft. Number of drillers and helpers per shift: 2 drillers and 2 helpers. Number of drill shifts per day: 1. Explosive: 40 per cent gelatin dynamite. Number of muckers per shift: 3 Number of mucking shifts per day: 1. Type of haulage: Horses. Wages: Drillers, $4.50; helpers, $4; muckers and trammers, $3.50; blacksmith, $4.50. Maximum progress in any calendar month: 170 ft., August, 1904. Average monthly progress: 150 ft. (last 10 months).

COST OF DRIVING THE TUNNEL

	Feet.	Cost per foot of tunnel.
1901	12	$23.88
1901-2	190	22.98
1902-3	377	27.94
1903-4	702	21.99
1904-5	1,077	21.19
1905	292	30.37
Average for	2,950	$23.38

These costs include all labor, supplies, repairs, powder, fuse, caps, candles, tools, lubricants, and general expenses, and the total value of the electric-drill plant with which the tunnel was started, and the total value of the air-drill plant which succeeded it, together with tunnel buildings, pipe, rails, and the ventilator, with no credit for salvage on any of this permanent equipment. The fiscal year dated from Sept. 30. The tunnel was driven in 1901-3 with electric drills, and the high cost for 1905: 292 ft., $30.37.

STRAWBERRY TUNNEL.

Important Details.—Location: Utah and Wasatch Counties, Utah. Purpose: Irrigation and reclamation. Cross section: Straight bottom and walls, with arched roof. Size: 8 ft. wide by 9½ ft. high. Length: 19,100 ft. Character of rock penetrated: Limestone with interbedded sandstone, and sandstone with interbedded shale. Type of power: Electric power generated in a hydraulic plant operated in connection with the tunnel. Distance of transmission from west portal to power house approximately 23 miles. Ventilator: Pressure blower. Size of ventilating pipe: 14 ins. Drills: Piston pneumatic, usually 2 in the heading. Mounting of drills: Vertical columns. Number of holes per round: 16 to 18. Number of drillers and helpers per shift: 2 drillers and 2 helpers. Number of drill shifts per day: 3. Explosive: 40 per cent gelatin dynamite. Number of muckers per shift: 6. Number of mucking shifts per day: 3. Type of haulage: Electric after first 2,000 ft. Wages: Drillers, $3.50; helpers, $3.25; muckers, $2.75; motormen, $3.25; brakemen, $2.75; blacksmiths, $4; helpers, $2.75. Maximum progress in any calendar month: 500 ft., November, 1910. Average monthly progress: 320 ft. per heading.

COST OF DRIVING THE TUNNEL.

	Feet.	Cost per foot of tunnel.
West heading—		
Previous to 1909	1,613	$60.05
During 1909	3,892	33.58
During 1910	5,021	30.56
During 1911	3,491	41.52
January to July, 1912	2,382	36.79
East heading, October, 1911, to July, 1912	2,682	33.04
Average for	19,081	$36.78

DETAILED COST OF DRIVING THE WEST HEADING FOR THE YEAR 1909, 3,892 FT.

	Cost per foot of tunnel.
Labor—	
Engineering	$0.49
Superintendence	.73
Shift bosses	1.22
Timekeepers	.36
Drillmen and helpers	3.15
Miners (for handwork, trimming, etc.)	.23
Muckers	2.96
Track and dump men	.74
Mule drivers	.39
Motormen and brakemen	.44
Electricians and blower men	.07
Disabled employes	.19
Timbermen	.22
Miscellaneous	.40
	$11.59
Materials—	
Powder, fuse, caps, etc.	3.08
Lumber	.29
Oils, candles, etc.	.22
Ventilating pipe	.64
Track, including ties	.63
Pressure air pipe	.49
Drill repair parts (including hose)	.18
Miscellaneous	.19
	$5.68
Repairs—	
Machine shop expense (including labor and supplies)	.93
Blacksmith shop expense (including labor and supplies)	1.22
	$2.15
Power (all purposes)	7.65
Depreciation—	
Haulage equipment	.09
General equipment	1.00
	$1.09
General expense	3.96
Camp expense	1.21
Corral expense	.25
	$5.42
Total	$33.58

"General expense" includes a proportionate charge for the expenses of the Provo office, such as salaries, stationery, telephone, and supplies; also a proportionate charge for the expenses of the Washington, the Chicago, and the supervising engineer's offices. The Provo office covers approximately 68 per cent of this charge, the Washington office 23 per cent, the Chicago office 2 per cent, and the supervising engineer's office 7 per cent.

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*A more detailed statement of the cost of ls tunnel may be found in an article entitled A Problem in Mining, Together With Some ıta on Tunnel Driving," by F. M. Simmons ıd E. Z Burns, Bull. Am. Inst. Min. Eng., arch, 1913, p. 369.

DETAILED COST OF DRIVING THE WEST HEADING FOR THE YEAR 1910, 5,021 FT.

	Cost per foot of tunnel.
Labor—	
Engineering	0.61
Superintendence	.69
Shift bosses	1.25
Timekeepers	.22
Drillmen and helpers	2.85
Miners	.28
Muckers	2.93
Track and dump men	.71
Motormen and brakemen	1.49
Electricians and blower men	.13
Disabled employes	.16
Timbermen	.28
Miscellaneous	.07
	$11.58
Materials—	
Powder, fuse, caps, etc.	3.52
Lumber	.22
Oils, candles, etc.	.20
Ventilating pipe	.65
Track, including ties	.74
Pressure air pipe	.28
Drill repair parts (including hose)	.24
Miscellaneous	.07
	$5.92
Repairs—	
Machine shop expense (including labor and supplies)	.90
Blacksmith shop expense (including labor and supplies)	1.23
	$2.13
Power (all purposes)	5.70
Depreciation—	
Haulage equipment	.20
General equipment	1.00
	$1.20
General expense	3.32
Camp expense	.63
Corral expense	.08
	$4.03
Total	$30.56

DETAILED COST OF DRIVING THE WEST HEADING, FOR THE YEAR 1911, 3,419 FT

	Cost per foot of tunnel.
Labor—	
Engineering	$0.45
Superintendence	.82
Shift bosses	1.65
Timekeepers	.38
Drillmen and helpers	4.07
Miners	.37
Muckers	5.13
Track and dump men	2.00
Motormen and brakemen	1.87
Electricians and blowermen	.08
Disabled employes	.48
Timbermen	1.72
Miscellaneous	.05
	$19.07
Materials—	
Powder, fuse, caps, etc.	2.61
Lumber	.80
Oils, candles, etc.	.43
Ventilating pipe	.77
Track, including ties	1.52
Pressure air pipe	.36
Drill repair parts (including hose)	.34
Miscellaneous	.25
	$7.08
Repairs—	
Machine shop expense (including labor and supplies)	2.16
Blacksmith shop expense (including labor and supplies	1.54
	$3.70
Power (all purposes)	5.20
Depreciation—	
Haulage equipment	1.85
General equipment	.50
	$2.35
General expense	3.00
Camp expense	1.10
Corral expense	.02
	$4.12
Total	$41.52

DETAILED COST OF DRIVING THE WEST HEADING, JANUARY TO JULY, 1912, 2,382 FT.

	Cost per foot of tunnel.
Labor—	
Engineering	$0.36
Superintendence	.56
Shift bosses	1.08
Timekeepers	.26
Drillmen and helpers	3.08
Miners	.43
Muckers	1.95
Track and dump men	1.55
Motormen and brakemen	1.34
Electricians and blowermen	.18
Disabled employes	.18
Timbermen	2.59
	$16.85
Materials—	
Powder, fuse, cap, etc	2.72
Lumber	2.13
Oils, candles, etc	.32
Ventilating pipe	.70
Track, including ties	1.51
Pressure air pipe	.30
Drill repair parts (including hose)	.32
Miscellaneous	.39
	$8.39
Repairs—	
Machine shop (including labor and supplies)	1.39
Blacksmith shop (including labor and supplies)	1.02
	$2.41
Power (all purposes)	3.75
Depreciation—	
Haulage equipment	2.20
General equipment	.50
	$2.70
General expense	1.90
Camp expense	.79
	$2.69
Total	$36.79

DETAILED COST OF DRIVING THE EAST HEADING, OCTOBER, 1911, TO JULY, 1912, 2,682 FT.

	Cost per foot of tunnel.
Labor—	
Engineering	$0.19
Superintendence	.77
Shift bosses	1.36
Timekeepers	.31
Drillmen and helpers	3.62
Muckers	4.63
Track and dump men	2.00
Mule drivers	.89
Timbermen	1.80
Electricians and blowermen	.30
Disabled employes	.09
Miscellaneous	.21
	$15.87
Materials—	
Powder, fuse, caps, etc	$2.67
Lumber	.93
Oils, candles, etc	.36
Ventilating pipe	.45
Track, including ties	.56
Pressure air pipe	.12
Drill repair parts (including hose)	.38
Miscellaneous	.21
	$5.68
Repairs—	
Machine shop expenses (labor and supplies)	.62
Blacksmith shop expenses (labor and supplies)	.65
	$1.27
Power (all purposes)	$3.21
Depreciation—	
Haulage equipment	.47
General equipment	1.02
	$1.49
General expenses	1.86
Camp expenses	1.35
Corral expenses	.95
	$4.16
Pumping (labor and material)	1.36
Total	$33.04

City Managers—Some of Their Problems and Their Coming Convention.

(Contributed.)

Arrangements have been made by Mr. Charles E. Ashburner, city manager of Springfield, Ohio, for a conference of the City Managers of the United States on Aug. 4-5-6, 1914. This convention will be the first of its kind ever held.

Springfield is one of the cities governed by this new plan of municipal government—one of the first ones in the United States to adopt the new form—and the delegates will doubtless be cordially received. They can make a study of the results already attained in the city where the innovation has been in practice the longest time.

There are many good points in favor of a conference of this body. Since the profession is an absolutely new one, with reference to municipal control, it is reasonable to suppose that there is need of organization. With unison in effort the chances for success are much enhanced. At present every city has a different idea as to just what the duties of their manager or superintendent should be, and each of the managers no doubt finds it quite puzzling at times to know just what is expected of him and just how much power he may assume he is entrusted with.

To make this new scheme of municipal business management a success there must be an existence of harmony between the governing body and its manager. Therefore, if there could be some definite understanding as what requirements should be set forth by t different cities, then it would be easier for t manager to carry out the plan. It is an app rent fact that there will never be an existen of harmony if these matters are not settled.

Many of the present managers, no dou are now working along indefinite lines. Th may have sufficient power to carry on the cit affairs as they may see fit, but these pow are at all times too well under the electi body's control. Differences in opinion w doubtless arise and so long as the manager h to depend upon the commission or council i the occupancy of his office, he may doubt ju what independence he possesses.

No matter what the charter may be, und which the plan is operated, there must harmony, and when this condition exists t executive body will not hesitate to allow t manager full control of the city's affairs.

Suppose, however, that we imagine such charter as would make it compulsory for t elective body to surrender most of their a thority to a manager, then under these con tions imagine a fair, representative governi body with an unfair person as manager. T results would be hazardous and the new fo with all of its good merits would be co demned.

Granting that good legislative measures c ating a charter, defining the requirements this new form, were enacted, would it be pra ticable for the different states to lay down su rules as would regulate all of its municipalit in one general charter? It seems impossil to do this without affecting certain localities their detriment.

The conditions in every locality are su that the privilege of regulating the manage duties should be put under the power of inhabitants. For example, a community havi an efficient engineering department and un conditions which would favor a compet person as general supervisor of purely busin matters, such as promoting and encourag movements of industry, or the general devel ment of civic conditions, would call for combination of the city's governing body w the chamber of commerce, and a person the head known as city supervisor.

On the other hand, consider a municipal with no particular possibilities for devel ments of industries, nor deeming it necess to co-operate with the chamber of comme Here we have the most common state of aff and the one which seems to fit the majo of places. For this case we need a pers fitted to advise the commission or council all matters pertaining to the city's busin and one who can carry out their acts with pert judgment. And since practically all of city's business is of an engineering nature, person should be an engineer and defined as engineer-manager.

It is very probable that such questions suggested are already being measured by m of the different cities, municipal leagues, all who are interested in better municipal g ernment; all with the possible idea of introd ing bills before the next legislature to regu these matters.

With the seeming conditions of confus now existing it seems very necessary for s sort of meeting, before undesirable action be taken which will only retard the growth the new movement instead of stimulating i

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Overhead wires at Richmond, Indiana, carried on pole lines of which every sev pole is concrete. A 25-ft. pole, 11 ins. squ at the butt and 6 ins. square at the top, c $7.65, not much more than a good cedar when dressed and treated ready for sett The setting cost is somewhat heavier than wood, but the advantage of positive ancho of the line at short intervals more than off the added cost. During a recent storm a ft. concrete pole, anchored in two directi was so severely strained by the fallen lin both sides that one anchor was pulled enti out of the ground. When the strain was lieved the pole went back into place with apparent harm.

Roads and Streets

A Cost Accounting System for Contract, Convict and Hired Labor Forces Used in Alabama.

Contributed by R. P. Boyd, Assistant State Highway Engineer of Alabama.

The accounting system outlined in this article has been used by the Alabama Highway Commission for two years. While not so complete in some of its details as those in use by other states it fits southern labor conditions and the types of road generally constructed in Alabama. Little difficulty has been experienced by counties and foremen in understanding and conforming to its details. This system is satisfactory in Alabama and I believe it can be used, with slight necessary modifications, in any county in the South.

ORGANIZATION FOR STATE AID IN ALABAMA.

The organization employed on state aid work (some knowledge of which is necessary to understand the system of cost keeping and accounts) is, briefly, as follows:

On state aid work, one-half of the cost of which is paid by the state, resident engineers are employed who are paid jointly by the state and county. Construction work may be done by contract, force account or convicts, or by a combination of them all The work is under the supervision of the resident engineer who is the field accountant, making up reports which are submitted to the chief engineer and secretary of the commission, W. S. Keller, who returns them to him after approval. The accounts are paid by the county out of a joint state and county fund. Duplicates of accounts are retained by the commission.

The road to be improved is carefully surveyed and the foreman furnished a profile or plan showing the work in detail. The yardage of all cuts and fills and points where borrow pits are needed are shown. The size and location of each culvert and other information of use to the foreman is given. Quantities for force account or convict work should be as accurate and plans as carefully prepared as for work done by contract.

ACCOUNTING SYSTEM.

Accounting is by a modified voucher system. Cost keeping is separate from the accounts yet is dependent upon them for material and labor totals.

It will be noted that by determining at suitable intervals (usually once a month) from the various bills and labor accounts the cost per day of men and teams and using these costs in determining unit costs a distribution of lost time, overhead charges and incidentals is effected.

In using the various cost keeping forms illustrated the rate per day of men and teams are not filled in until these daily costs are determined at the end of the month, in the case of convict labor and, should it be desirable, also for force account work. The cost of contract work is usually taken from the estimate sheets, although it may be determined accurately as described later.

This system of accounting and cost keeping may easily be elaborated to include unusual items but an effort has been made to keep it simple in its details. However, simple as it is the costs are sufficiently accurate for all practical purposes.

Force Account Work.—The foreman is furnished a 15-week time book, 4¾x8 ins. in size, Fig. 1, arranged to show the number of men, the rate per day of each man and the point on the road at which they are working. In addition to this an individual weekly time sheet is kept by the foreman, Fig. 2, showing each man's name, the character of his work, the number of days worked, and the amount earned during the week. These are made in pad form with carbon and must be sent to the resident engineer in duplicate. At the end of the week the foreman makes out a report, Fig. 3, which gives in detail the work accomplished during the week, between what stations, and the cost of it. This is sent with the pay roll, Fig. 4, which has all the bills incurred during the period of the pay roll attached thereto. The resident engineer then makes up the week's pay roll, Fig. 4, which shows the whole cost of labor taken from the form, Fig. 2; and the cost of supplies, taken from bills rendered. He sends the form, Fig. 2, and bills in duplicate attached to the form, Fig. 4, to the chief engineer of the commission. On the back of the form, Fig. 4, or on a separate sheet, the resident engineer shows an estimate of the work done during the period of the pay roll and the cost of each class of work. When the pay roll is received in the office of the commission, it is checked by the bookkeeper and, if correct, is approved and returned to the county for payment. The bookkeeper then posts his book, which is a loose leaf ledger, Fig. 5b. This ledger shows each class of work; the amount of work accom-

For Week Ending *April 18, 1914*	Sun.	Mon.	Tues.	Wed.	Thur.	Fri.	Sat.	Wagesy per Da	Total
5 Men Clearing and Grubbing		*50*	*50*	*50*				*$0.70*	*$10.50*
3 Men Clearing and Grubbing					*30*	*30*	*15*	*0.70*	*3.55*
2 Teams Clearing and Grubbing									
Clearing and Grubbing has been done from Sta. to Sta. *50+00-70+50*									
10 Men Grading		*100*	*100*	*100*			*50*	*0.70*	*23.50*
12 Men Grading					*120*	*120*		*0.70*	*16.80*
Men Grading									
6 Teams		*60*	*40*	*60*	*60*	*60*	*30*	*2.11*	*65.50*
Grading has been done between Stations *25+00-30+50*									
Men Surfacing									
3 Men Surfacing			*30*	*30*	*30*	*30*	*15*	*0.70*	
Men Surfacing									
3 Teams Surfacing (*Sand clay, av. 50′ haul*)			*30*	*30*	*30*	*30*	*15*	*0.211*	
Teams Surfacing									
Hours Road Machine used (*3 teams and 3 men*)		*8*	*12*	*10*	*10*	*10*		*8.43*	*42.15*
Hours Roller used									
Gasoline or Coal used									
Surfacing has been done between Stations *8+00 20+00*									
2 Teams Hauling for Culverts or Bridges *#10*		*20*	*20*					*2.11*	*8.44*
Men working on Culverts or Bridges									
8 Men Working on Culverts or Bridges *#10*		*80*	*80*						
3 Men Working on Culverts or Bridges *#10*						*30*		*2.11*	*6.33*
12 No. cu. yds. Gravel or Sand used on Culverts or Bridges *#10*									
168 No. sacks Cement used on Culverts or Bridges *#10*									
No. ft. B. M. Lumber used in Bridges or Forms *#10, 500 new*									
No. lbs. Steel used in Bridges or Culverts									
At what Stations were Culverts or Bridges put in *25+80, 30+00, 42+00*									
No. ft. 12″ Pipe put in									
30 No. ft. 15″ Pipe put in— *#12 Cor. 2 men*				*20*				*0.70*	*1.40*
No. ft. 18″ Pipe put in — *1 team*				*5*					*1 05*
33 No. ft. 24″ Pipe put in— *#13 Av. 2 men*					*20*				*1.40*
Extra Work — *1 team* (Extends across two pages)					*5*				*1.05*

STATE
HIGHWAY DEPARTMENT
Foreman's Memorandum

..............County.
..............Road.
..............Type of Road.
.............Width of Roadbed.
.............Width Surface.
..............Engineer.
..............P. O.
..............Foreman.
..............P. O.

INSTRUCTIONS.

Use this book to keep each class of work separate. On Saturday night make out your report on the Foreman's Weekly Report Blanks and mail to the Resident Engineer in charge of your work.

Neglect to keep a complete record as indicated in the following blanks will be sufficient cause for removal.

This book must be returned to the State Highway Department upon the completion of the work.

(Front page of book 4¾″x8″ in size)

Fig. 1. Typical Pages from Foreman's Memorandum Book.

STATE AID ROAD
WEEKLY TIME SHEET
.......*Fayette*.. ....COUNTY For Week Ending..........*April 18*......1914

NAME	Position	Sun.	Mon.	Tues.	Wed.	Thur.	Fri.	Sat.	No. Days Worked	Rate Per Day	Amount Due	SIGNATURE
J. Jones	*Foreman*									*Month* *$100 00*	*$100.00*	
A. Smith	*Guard*			*Month of April*						*30 00*	*30 00*	
B. Brown	*Guard*									*30 00*	*30.00*	
K. Williams	*Guard*									*30.00*	*30.00*	

(Original form 8½″ x 8½″ in size)

										Total	*$190.00*	

Approved:
.... *W. R. K* ....
Resident Engineer.

The above account is true and correct.
....................*J. Jones*..................
Foreman.

Fig. 2. Time Sheet for Hired Labor Prepared by Foreman.

plished being written in black ink and the cost of it in red ink. At any time a glance at the ledger shows the work in detail and the cost of each separate item.

Contract Work.—On contract work the engineer keeps track of the work by cross-sections, taking estimates and measuring up work each month. An estimate sheet, Fig. 6, is used, on the reverse side of which a distribution is made as illustrated. The items of the estimate and distribution are recorded in the office of the commission in a ledger, the form of tabulation being shown in Fig. 5a.

Actual costs are not determined. The method of keeping convict and force account labor costs might be applied to their work with satisfactory results, but would require co-operation between the contractor and engineer.

In Fig. 7 is shown the form of payroll for engineering services in use on all classes of work. A filing record 3 ins. wide is printed in a fold of the reverse side. This form is made out in triplicate and handled in a manner similar to the monthly estimate.

Convict Work.—When convict forces are used the work is reported in practically the same manner as on force account work except that costs are figured by the engineer at the end of the month. In this case, foremen and guards are generally the only paid men. The convict counties pay off monthly and all bills and expenses are filed before the engineer can tell at what rate per day the convicts and teams will be charged.

In computing this the total of the number of men reported daily by the foreman is used. For example, say there are 25 convicts that have worked for 26 days, then there are 650 convict-days, the cost of maintenance being as follows: One-half foreman's salary $50.00 (half charged to teams and half to men); salary of three guards at $30.00; feeding, clothing, drugs, medical attention, etc., $315.00; a total of $455.00 at a cost of $0.70 per day per convict.

Suppose there are 10 teams that have worked 26 days, then there are 260 team-days for the month at a cost of, say, one-half salary foreman, $50.00, cost of feed, blacksmithing, repairs, etc., $500.00; total $550.00, or a cost of $2.11 per day.

The resident engineer turns in all bills with the form, Fig. 4, and the rate per day is checked in the office. From the forms, Fig. 3 (only one week's work is illustrated), the number of men and teams on each class of work is obtained and the cost is figured, in this case, $0.70 per day for each man and $2.11 per day for each team. The pay roll is recorded in the ledger, Fig. 5a, as described before.

Combined Work.—Where work is performed by a combination of these methods the same forms are used, separate accounts being provided for each.

STATE AID ROAD

FOREMAN'S WEEKLY REPORT

Fayette County. Week Ending *April 18* 1914
Camp. Post Office. Foreman *J Jones*

CLEARING AND GRUBBING

5	men	*25*	days @ $	*0 70*	per day	$	*10 50*
3	men	*4½*	days @ $	*0 70*	per day	$	*3 15*
	men		days @ $		per day	$	
	teams		days @ $		per day	$	
					Total	$	*13 65*

Clearing and grubbing has been done between Station *50+00* and Station *70+50*.

GRADING

10	men	*35*	days @ $	*0 70*	per day	$	*24 00*
12	men	*24*	days @ $	*0 70*	per day	$	*16 80*
	men		days @ $		per day	$	
	men		days @ $		per day	$	
6	teams	*30*	days @ $		per day	$	*65 50*
	teams		days @ $		per day	$	
					Total	$	*106 30*

Grading has been done between Station *25* and Station *30+50*

SURFACING

3	men	*13½*	days @ $	*0.70*	per day	$	*9 45*
	men	*13½*	days @ $	*2.11*	per day	$	*28 50*
	men		days @ $		per day	$	
	Roller men		days @ $		per day	$	
	Machine teams	*5*	days @ $	*8 43*	per day	$	*42 15*
	teams		days @ $		per day	$	
	Coal		tons @ $		per ton	$	
	Gasoline		Gals. @ $		per gallon	$	
					Total	$	*80.10*

Surfacing has been done between stations *8* and station *20*

CULVERTS AND BRIDGES

8	men	*16*	days @ $	*0.70*	per day	$	*11 20*
3	men	*3*	days @ $	*2.11*	per day	$	*6 33*
	men		days @ $		per day	$	
2	Teams hauling	*1*	days @ $	*2.11*	per day	$	*8 44*
					Total	$	*25.07*

#10

No. cu. yds. gravel, sand or rock used in culverts	*12*	@ $	*0.33⅓*	$	*4 00*
Number sacks cement used	*108*	@ $	*0.40* per sack	$	*43 20*
Number feet B. M. lumber used in bridges or forms	*500*	@	*20.00*	$	*10 00*
Number pounds reinforcing or steel beams used				$	
Culvert work has been done at stations	*25+80, 30+00, 42+00*				
No. feet 12-inch pipe put in		@ $	per lin. foot	$	
No. feet 15-inch pipe put in	*30*	@ $	per lin. foot	$	*2 45*
No. feet 18-inch pipe put in		@ $	per lin. foot	$	
No. feet 24-inch pipe put in	*33*	@ $	per lin. foot	$	*2.15*
			Total	$	*4 00*

Pipe has been put in at Stations *30+00, 42+00*

FORCE ACCOUNT

	men		days @ $		per day	$	
	men		days @ $		per day	$	
	men		days @ $		per day	$	
	teams		days @ $		per day	$	
					Total	$	

Force account has been done at Stations.

Fig. 3. Foreman's Typical Weekly Summary of Work.

Tunnel in Berlin. The construction of a tunnel under Berlin's most noted avenue, "Unter den Linden," has been proposed to relieve congestion of traffic. The plan includes parallel eastern and western sections, with one northern entrance for four street car lines, and two southern entrances for two lines each.

Concrete driveways for traction engine use are being built throughout the yards and shops of the Avery Co. at Peoria, Illinois. The concrete is 15 ins. thick and the edges are protected from spalling by a 2x8-in. plank laid flush with the surface of the road and bolted in place. Concrete was used four years ago for floors in the erecting and testing shops, and these floors are still good for many more years of service.

Statistics and Methods of Financin Repaving.

The following abstract from a report the Municipal Committee of the Clevelar Chamber of Commerce gives statistics methods of financing repaving with various in teresting recommendations in regard theret

Under the charter of the city of Clev land, the city is only required, at the optic of the city council, to pay for new pavemen and for repaving (after a pavement has bee down 15 years) 2 per cent of the cost, pl the cost of street intersections. The prese problem is largely one of repaving and r pairing. It has been customary in the pa for the city to pay 50 per cent of the cost repaving, the owners of property benefit paying the balance. By decreasing the pe centage which the city is required to pa the city council can shift the burden fro the city to the property owner.

The methods of apportioning the cost pavements in fifty of the important cities this country are as follows:

New Paving—
In 62 per cent of these cities the property pa all.
In 6 per cent the property pays 50 per cent.
In 10 per cent the property pays more than per cent and less than 100 per cent, exce in one city, where the property pays 30 p cent.
In 22 per cent of these cities the city pays a
Repaving—
In 42 per cent of these cities the property pa all.
In 10 per cent the property pays 50 per cent.
In 8 per cent the property pays between and 100 per cent.
In 40 per cent the city pays all.

Approximately 25 per cent of the citi pay the major part of the original pavin while 40 per cent of the cities do the maj part of repaving.

Upon certain down town streets where t business is of such a nature that the con tion of the streets is an important fact the proportion paid by the property own might well be increased. This might also the case upon certain main thoroughfar where additional benefits are derived fro greater traffic. In such instances the c might well be relieved of a considerable pa of the cost of repaving. For other stre the cost should be shared in just proporti probably under the old method of 50 cent for the city and 50 per cent for the pr erty owner. There could be little objecti to the city issuing bonds for its portion repaving provided such bonds were issued a term of years equal to the life of the pa ment.

We believe that repaving might be term a repair or a renewal of pavements, and t the city's proportion thereof, theoretica should be provided for from tax receip However, assuming the life of a pavement be 20 years, it would be impossible to aside from the city's revenue a suffici amount for this purpose. Cleveland has miles of pavements, the original cost of wh averaged, approximately, $60,000 per mile

If the average life of a pavement is years, an average of 25 miles must be paved annually, at an expense of $1,500,000 may be interesting to note that if the pol were pursued for 20 years of repaving miles per year, and 20-year paving bo were issued at the rate of 4½ per cent annum, the annual cost for sinking fund interest would be $2,340,000.

The repair of pavements is clearly an erating cost. It should be paid out of receipts. A survey of street conditions sho be made by the proper department, an report submitted to the proper city offic at the beginning of each year; the bo necessary for repaving should be issued mediately thereafter, and the funds repor as necessary for repairs during the y should be appropriated for that purpose.

As street paving is without doubt one the most important municipal functions, i the plain duty of city officials to keep pavements in good condition. We bel that if funds are not available for all of city activities which are being assum it is necessary for the city, as it w

STATE AID No. *20*
.... *Fayette* .... COUNTY

LABOR PAY ROLL
.... *Fayette*..........COUNTY, DR.
TO ......*William Childers*........
FOR LABOR PAY ROLL

NAME	TITLE	Period of service inclusive From	To	No. Days	At Per Day	Amount Due	Amount Received	Signature
For labor, per attached pay roll						*$190.00*		
J. J. Gracery, per attached bills						*200.00*		
Clothing Co., per attached bill						*85.00*		
Drug Co., per Attached bill						*15 00*		
Dr. A., per attached bill						*15.00*		
Grain Co., feed, per attached bill						*420 00*		
Blacksmithing repairs, per attached						*20.00*		
Hardware, shoes, nails, etc						*60.00*		

CONVICTS

Foreman	*$50.00*
Guards	*90.00*
Groceries	*200.00*
Clothing	*85.00*
Drugs	*15.00*
Doctor	*15.00*

650) $455.00(70c per day convict.

TEAMS

Foreman	*$50.00*
Feed	*420 00*
Repairs	*20.00*
Hardware	*60.00*

260) $550.00 (2.11 per day team.

TOTAL *$1005.00*

I hereby certify that this pay roll is correct and the amounts are correct, due and unpaid.
................*W. R. K*..............
Resident Engineer.

Approved for payment
.........*W. S. K*.........
State Highway Engineer.

(Original form 9½" x 15" in size)

STATE AID
......*Fayette*..........County

LABOR PAY ROLL
Month of ....191..

DISTRIBUTION

	Dolls.	Cts
Grading 2000 cu. yds	*405*	*00*
Clearing and grubbing 1 acre	*40*	*00*
Gravel surfacing 400 cu. yds	*250*	*00*
Sand-Clay surfacing 800 cu. yds	*200*	*00*
Culverts	*110*	*00*
Total	*1005*	*00*

NOTE

Resident engineers must make these pay rolls out in triplicate at the end of each month or at the termination of work in the County, and mail original and duplicate to the State Highway Engineer.

The duplicate will be retained in the office of the State Highway Department and the original, after being approved by the State Highway Engineer, will be returned to the Resident Engineer, who will present it to the proper county authority for payment.

(Note on second fold.)

(On reverse side, 3" wide, to fold.)

Fig. 4. Labor Payroll Prepared by Resident Engineer.

for a business firm or individual, to retrench; retrenchment should curtail those activities which are not primarily functions of municipal government, in order that those functions which are clearly the duty of the municipality may not be impaired.

Specifications for County Road Maps in Montana.

The various counties of the state of Montana are required to furnish to the state highway commission a county road map which conforms to the specifications given below:

Scale and Symbols.—All maps shall be made upon tracing linen in sheets 34x37 ins. over all. Scale shall be 1 mile per inch and shall show all government and legal sub-divisions. State line to be heavy black line of dashes and two dots. County lines to be heavy black dashes and one dot. Reservation lines to be heavy dashes or broken line in black. Mountain crests and important divides may be shown where advisable in broken black lines. Meridian, parallel, township, range and section lines in full black, varying in width with the importance of the line shown. Quarter section lines, light broken black lines. Subdivisional lines, light black dots or dashes. Railroads, full black lines hatched. Telephone lines, black dots and dashes. Lakes shown in blue lines shaded. Canals in light black lines, lighter than natural water courses. Streams in full blue lines. Dry streams, coulees, etc., in dotted blue lines. Land marks may be shown where desirable. State highways shown in broken red filler. Main highways shown in broken yellow filled. Towns shown hatched in black in proper legal subdivisions or fractions. Common highways, double lines, spaced 1-50 in. Location of road material shown by red circle 1-8 in. in diameter with consecutive numbers inside if more than one on a sheet, and character and extent of material shown in legend. Surveyed roads, full double line 1-50 in. space. Unsurveyed roads, broken dotted line 1-50 in. space. Bridges 10-foot span or over designed (thus |20′ C) with initial of material and span in feet in fine letters. |100 S T would indicate a 100-

COUNTY... *Winston*.... ... AID FOR 191*4*..
CHARACTER ROAD ....*Gravel* ....WIDTH ROAD BED .*20* .Ft. WIDTH SURFACE..*12* Ft. LENGTH SURVEY..*30*..MILES
ENGINEER.... .*M. S. Bingham*. .... CONTRACTOR..... *G. B. Edgar*...... DATE OF CONTRACT....*Oct. 1, 1913*.... FINISHED ROAD......MILES

No. of Pay Roll	Date	Excavation—Cubic Yards Earth	Loose Rock	Solid Rock	Clearing and Grub. Acres	Pipe—Lineal Feet 12"	15"	18"	20"	24"		Culverts and Bridges Concrete Plain	Concrete Reinforced		Lumber Ft. B. M.	Retaining Walls Cu. Yds.	Rip Rap Cu. Yds.	Surface Cu. Yds.	Cu. Yds.	Avg. Haul	Force Acct.	Extras	Engineering Preliminary	Construction	Total Amount of Estimate	Amount Paid	Amount Retained	Appropriation Date	State or County	Amount	Remarks
		372.22			*47.50*			*38.50*		*109.37*						*3150*		*365.75*													
3	*2 1*	*1691.9*			*½*			*2.7½*		*62½*						*7.0*		*770.00*		*½ mile*				*165.70*	*964.84*	*820.12*	*144.72*				

(Original form 16¾ in. by 27 in. in size.)

(a)

COUNTY.............................. AID FOR 191..
CHARACTER ROAD .... WIDTH ROAD BED......Ft. WIDTH SURFACE......Ft. LENGTH SURVEY......MILES
ENGINEER.................... ... CONTRACTOR....... .... DATE OF CONTRACT.............. FINISHED ROAD........MILES

No. of Pay Roll	Date	Excavation Cu. Yds. Earth	Loose Rock	Solid Rock	Clearing and Grub., Acres	Pipe—Lineal Feet 12"	15"	18"	20"	24"	Culverts and Bridges Concrete Plain	Concrete Reinforced	Lumber, Ft. B. M.	Retaining Walls, Cu. Yds.	Rip Rap, Cu. Yds.	Surface Gravel, Cu. Yds.	Sand Clay, Cu. Yds.	Avg. Haul	Force Acc't	Extras	Engineering Preliminary	Construction	Maintenance of Outfit No. Teams	Feed Bills	Repairs	Incidentals	No. Men	Provisions	Extras	Extras	Teams Rate Per Day	Rate Per Day	Men				Total Amount of Pay Roll	Appropriation Date	State or County	Amount
		40500			*10*						*70*		*40*			*250*	*200*																							
20	*1 18*	*2000*			*1*						*10*		*2000*			*400*	*800*	*2 miles*				*100*	*10*	*420*	*20*	*60*	*25*	*200*	*30*	*85*	*211*		*.70*				*1005*			

(Original form 16¾ ins. by 27 ins. in size)

(b)

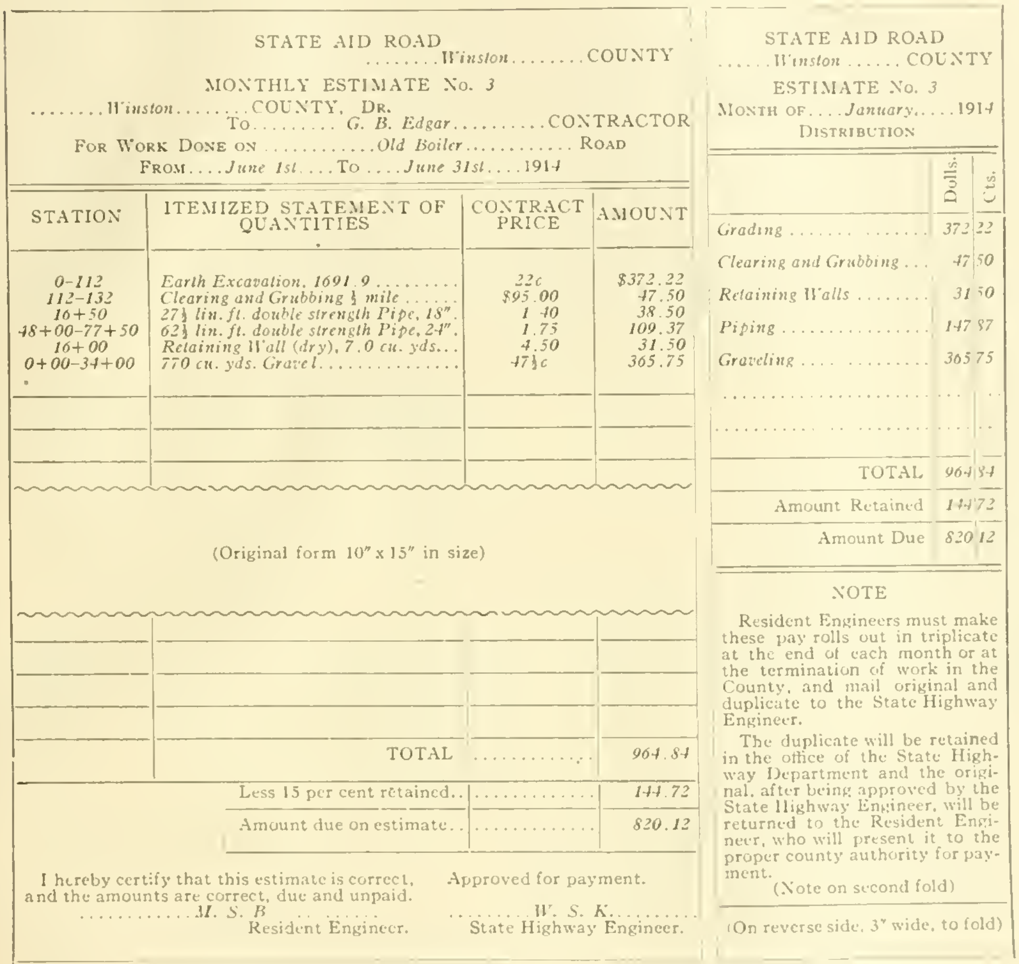
Fig. 5. Form of Permannt Record Made by State Highway Commission.
(a) Contract work; (b) Convict and force account work.

ft. steel truss bridge. "C" would indicate concrete, "W" wood.

Lettering.—Upper case, capitals as A, B, C, etc., and lower case a, b, c, etc. All reservation letters to be 3-16 in. shadow letter, all capitals. Meridian, parallel, township and range, lettering to be capitals, and lower case; capitals and numbers to be 3-16 in. high and lower case letters 1-8 in. high. Section numbers to be 1-8 in. high. Lot and fractional numbers to be 1-16 in. high. Rivers to be caps and lower case letters; caps 3-16 and lower case 1-8 in. Tributary streams, caps and lower case; caps 1-8 in. and lower case 3-32 in. Unincorporated towns, caps and lower case; caps 1-8 in. and lower case 3-32 in. Incorporated towns all caps 1-8 in. Important land marks, mountain, peaks and divides, all caps and lower case; caps 3-32 in. and lower case 1-16 in. Railroads all caps 3-32 in. high. Canals, caps and lower case; caps 1-16 in. and lower case to conform. Telephones, caps and lower case; 1-16 in. and lower case to conform. Lakes lettered as rivers, tributaries or land marks according to their size and importance. Letters to be plain block type known as the "Eng. News" style; names all in caps to be vertical and those having caps and lower case letters to be inclined at an angle of 30° or 3 to 8.

Index Sheet.—Where the size of a county takes more than two of the sheets above specified, a small skeleton outline of the county should be made, showing the boundaries of each sheet by township and range, indexing the sheets. Vandyke prints with skeleton sketch for index, colored to indicate designated highways will be furnished Highway Commission. When Vandyke prints are to be furnished all lakes and streams will be put on tracing in black line.

Motor Bus Line in Asia.—A motor bus service is proposed across Asia Minor from Beirut to Bagdad. The busses will carry mail and passengers and will travel over a desert road made by horse and camel traffic. The trip will be shortened from 20 days to one week or less.


STATE AID ROAD
........Winston........COUNTY

MONTHLY ESTIMATE No. 3
........Winston........COUNTY, Dr.
To......... G. B. Edgar..........CONTRACTOR
For Work Done on ....Old Boiler............ Road
From....June 1st....To....June 31st....1914

STATION	ITEMIZED STATEMENT OF QUANTITIES	CONTRACT PRICE	AMOUNT
0-112	Earth Excavation, 1691.9	22c	$372.22
112-132	Clearing and Grubbing ½ mile	$95.00	47.50
16+50	27½ lin. ft. double strength Pipe, 18".	1.40	38.50
48+00-77+50	62½ lin. ft. double strength Pipe, 24".	1.75	109.37
16+00	Retaining Wall (dry), 7.0 cu. yds...	4.50	31.50
0+00-34+00	770 cu. yds. Gravel...............	47½c	365.75

(Original form 10" x 15" in size)

	TOTAL	....	964.84
	Less 15 per cent retained..	....	144.72
	Amount due on estimate..	....	820.12

I hereby certify that this estimate is correct, and the amounts are correct, due and unpaid.
.............M. S. B ....
Resident Engineer.

Approved for payment.
.........W. S. K.........
State Highway Engineer.

STATE AID ROAD
......Winston......COUNTY
ESTIMATE No. 3
Month of....January.....1914
Distribution

	Dolls.	Cts.
Grading	372	22
Clearing and Grubbing ...	47	50
Retaining Walls	31	50
Piping	147	87
Graveling	365	75
....		
....		
TOTAL	964	84
Amount Retained	144	72
Amount Due	820	12

NOTE

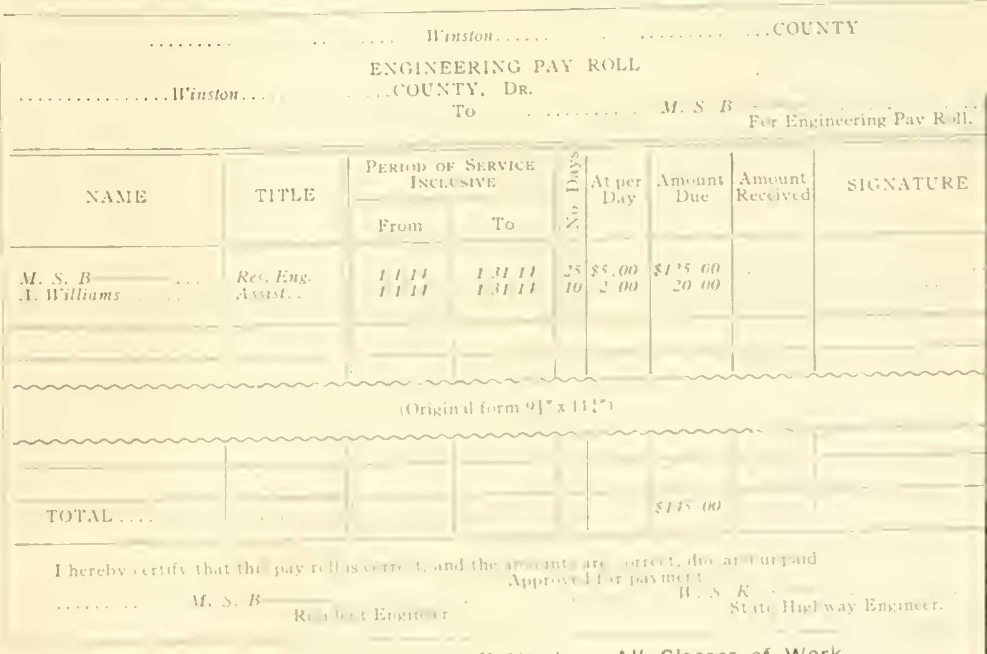
Resident Engineers must make these pay rolls out in triplicate at the end of each month or at the termination of work in the County, and mail original and duplicate to the State Highway Engineer.

The duplicate will be retained in the office of the State Highway Department and the original, after being approved by the State Highway Engineer, will be returned to the Resident Engineer, who will present it to the proper county authority for payment.

(Note on second fold)

(On reverse side, 3" wide, to fold)

Fig. 6. Monthly Estimate Sheet Used on Contract Work.


.........Winston..........COUNTY

ENGINEERING PAY ROLL
..................Winston......COUNTY, Dr.
To ....M. S. B
For Engineering Pay Roll.

NAME	TITLE	Period of Service Inclusive		No. Days	At per Day	Amount Due	Amount Received	SIGNATURE
		From	To					
M. S. B ...	Res. Eng.	1 1 14	1 31 14	25	$5.00	$125 00		
A. Williams ...	Assist..	1 1 14	1 31 14	10	2 00	20 00		

(Original form 9½" x 11½")

TOTAL....						$145 00		

I hereby certify that the pay roll is correct, and the amounts are correct, due and unpaid
.... M. S. B
Resident Engineer

Approved for payment
W. S. K
State Highway Engineer.

Fig. 7. Engineering Payroll Used on All Classes of Work.

Street Sprinkling and Oiling in St. Paul, Minn., in 1913.

The organization, equipment and cost of street sprinkling and oiling in St. Paul, Minn., is given in a recent report of the Commissioner of Public Works of that city. In our issue of June 5, 1912, and Jan. 15, 1913, will be found further data in regard to dust prevention by sprinkling in St. Paul.

The organization, under the sprinkling engineer, consisted of one general inspector, 10 district inspectors, with 8 to 15 teams each, one repair foreman with five men and two single teams, making repairs to standpipes and wagons, one record clerk and two assessment clerks.

The oiling organization consisted of one foreman, five double teams (oil and sand wagons) three attendants to the oil and sand wagons, one engineer and one night watchman at the oil plant.

The equipment in operation consisted of five oilers, four sand spreaders, 101 sprinkling wagons and one Studebaker automatic oiler 600-gal. capacity, costing $550; one Hvass Pillsbury oil distributor, 600 gal. capacity, costing $850; two Hvass sand spreaders, 1¼ cu. yds. capacity, costing $375 each and six Austin Western water sprinklers, capacity 750 gals., costing $245 each, added to the sprinkling equipment during the year; making a total of five oilers, four sand spreaders and 101 sprinkling wagons in service.

Water sprinkling began on March 31 and the last team was taken off Dec. 10. This is the longest sprinkling season yet recorded. 275¼ miles of streets were sprinkled of which 33 miles of macadam were oiled. As is it impractical to oil streets when they are damp or cold, oiling operations rarely begin before the last of May. These streets were therefore sprinkled with water during the first part of the season.

The unit costs of sprinkling and oiling for 1912 and 1913 are as follows:

Item.	1912.	1913.
Water sprinkling (2 trips per day) per front foot..........	$0.02775	$0.029
Oiling per application, per sq. yd.	0.0140	0.01

These increase in costs were influenced by the facts that sprinkler drivers were increased in pay from $100 to $115 per month, and the price of oil increased from $0.035 to $0.037 per gallon. The total amount expended on dust prevention was as follows:

Item.	Total co
Water	$ 99,662.
Oil	16,053.
Calcium chloride....................	3,031.
Total	$118,707.

Table I is a tabulation of a test run made of the teams in one sprinkling district (No. 1). A test of this kind is made of each team of the ten districts, once or twice a year as required, to determine whether the route is well laid out and whether the idle travel is excessive and to determine the general efficiency of the teamsters.

These run-out tests are made by the inspectors and are in each case of one half duration, the inspector following the te and recording the streets traveled, the ti etc. These records are platted in the office 600-ft. scale maps showing lengths of bloc location of hydrants, etc., and from this pl ting the productive travel and idle travel are determined.

The proportion of time devoted to actual sprinkling by the sprinkling teamsters is given below. Idle time is unavoidable and consists of rainy days and days following when dirt streets can not be sprinkled without making them muddy.

Item.	Time.
Total days	19,542 = 651.4 team months
Total days worked	13,701 = 456.7 team months
Total days idle	5,841 = 194 8 team months

Percentage of idle time to total days, 29.9.

On May 13, 1913, a test was made of a Sauer auto sprinkler. This sprinkler was on ts way to St. Cloud, Minn., and was exhibited ere by the agents and the writer made a test of it on Marshall, Selby and Summit Aves. The cost of the machine was $6,300, tank capacity 1,260 gals. The average of these tests was as follows: Time to fill from standard 2-in. hydrants 7 minutes; time to unload 1,200 gals. traveling at a speed of 7½ miles, 8 minutes. Length of street sprinkled full width with one tank load, one mile. Allowing 5 minutes' loss of time to a tank load, 30 miles treets per day could be sprinkled, but it is vident that with experienced operators that t least 35 miles per day could be covered as he auto can be operated at a speed of 8 to 0 miles per hour with greater efficiency. As horse-drawn wagon now sprinkles 5 to 6 iles per day, this auto sprinkler would take he place of at least 6½ teams. The cost of perating one team, salary of teamster, repairs, interest and depreciation is $128 per onth; 6½ teams cost $832 per month.

TABLE 1.—TEST RUN OF SPRINKLERS IN DISTRICT NO. 1, ST. PAUL, MINN.

Date.	Route.	Tank.	Total miles traveled.	Miles of street sprinkled.	Idle travel. Time.	Idle travel. Miles.	Time actual sprinkling.	Time filling.	Full time 5 hrs.	No. of fillings.
June 3, 1913	1	63	6.934	5.447	51	1.487	2-15	1-32	4-38	18
May 5, 1913	2	42	7.589	5.419	1-00	2.170	2-31	1-39	5-10	23
May 28, 1913	3	31	6.261	3.740	1-15	2.521	1-57	1-31	4-43	17
May 29, 1913	4	33	5.704	4.814	24	.890	2-44	1-38	4-46	20
June 2, 1913	5	61	5.320	4.937	15	.383	2-34	1-49	4-38	22
May 31, 1913	6	6	5.049	3.049	36	1.062	2-18	1-36	4-30	20
May 30, 1913	7	7	5.491	4.161	48	1.333	2-07	1-40	4-35	20
June 6, 1913	8	9	6.375	4.813	49	1.562	2-17	1-40	4-46	21

The estimated cost of operation of the above otor sprinkler, salary of operators, interest, epairs, depreciation and gasoline would be 463 a month, making a saving on team prinkling of $369 per month or $2,583 for the prinkling season.

The above cost is arrived at as follows:

Item.	Cts. per mile
ost of tires	$0.065
asoline	.035
il and grease	.0075
epairs	.04
	$0.1475

	Per season month
nvestment, $6,300 at 5 per cent, $315	$ 45
aintenance, 14¾ cts. per mile, $7.25 a day	218
hauffeur, $80; leverman, $60	140
epreciation, 15 years' life, $420	60
Estimated cost of operation	$163

The city engineer of St. Cloud has operated his auto sprinkler during six months; 2,822 anks of water were distributed over 2,833 iles of streets using .37 gals. of gasoline per ile or 12½ gals. per day at a cost of $2.20. he motor sprinkler displaced five teams, but ave more frequent sprinkling than was done y the teams. The average sprinkling per day as 34 miles of street, full width and the average of saving over team hire was $20 a day r a total saving of $1,667 for the season.

The city of Denver, Colo., is using this same ake of auto sprinkler, and also the city of t. Louis, but of a larger size, namely, 1,460 al. capacity. This machine cost $7,300. The ayor of Denver states that one machine displaced 10 teams and is satisfactory on all lasses of streets.

Ohio is spending seven million dollars on tate roads this year, which is more than in he preceding eight years.

Notes On Roads and Streets.

Office and Field Equipment for County Road Superintendents in Illinois.—The following is the equipment recommended by the Illinois Association of Road Superintendents to be supplied by counties to road superintendents:

Office Equipment—Furniture and Supplies: (1) Desk containing drawers on left and vertical letter file on right; (2) revolving desk chair, draftsman's stool and office chairs; (3) typewriting machine; (4) table 3 ft. wide and 6 to 8 ft. long; (5) draftsman table 42x60 ins. with adjustable stand; (6) small drawing board 20x24 ins.; (7) tee square; (8) map filing cabinet; (9) sectional filing case containing sections for books, legal blanks, vertical letter files, card index and drawers; (10) county atlas, detail paper, tracing cloth, tracing paper, plate profile paper, waterproof drawing inks, pencils, erasers, thumb tacks and lumber crayon, stationery, record books and field books 4½ ins. by 7 ins., left hand page ruled ¼x¾ ins., right-hand page ruled ⅛x¼; (11) each county superintendent should furnish his own triangles, scales, protractor, drafting instruments and calculating devices.

Field Equipment: (1) Engineer's transit with level attachment; (2) ranging poles; (3) steel chain tape 100 ft. long; (4) chaining pins; (5) locke hand level; (6) Philadelphia leveling rod, sectional or extension; (7) metallic tape 50 ft. long; (8) gravel testing outfit (as used by Highway Commission); (9) one soil testing outfit for bridge sites.

Efficiency of Teams Hauling Freight from R. R. Warehouses.—In an address before the Electric Vehicle Association, Fred A. Hortter, car accountant of the Boston and Maine Railroad, stated that observation of the terminal yard teaming in the Boston & Maine railroad yards in Boston showed that in one day 10,264 horse drawn vehicles handled freight to the outbound freight houses. The total outward tonnage for the week amounted to 22,416 tons, which showed an average horse vehicle load of .36 of a ton each. The efficiency of a five-ton unit operating under this load factor is very low. In another observation in a trucking concern hauling approximately 180,000 tons of merchandise annually, the ratio of loaded and empty mileage of the horse vehicles was found to be 241:143, while the actual moving time represented but 22.95 per cent of the total day's work. Of course these figures show the actual conditions studied in one city during but a comparatively short period. Nevertheless they indicate a lack of efficiency. Additional supervision and intelligent regulation of the trucking business along the lines in which the railroad have worked at their handling of freight is needed.

A Poured Bitumen Sheet Asphalt Patch.—A method of patching sheet asphalt by filling in with stone and pouring bituminous over the tamped surface is given by W. M. Pullar in the Proceedings of the American Society of Civil Engineers, pp. 446, as follows:

About two years ago the writer made a new experiment in repairing sheet-asphalt pavements. The section of pavement to be repaired was very badly cracked, and contained three holes, each about 1 sq. yd. in area. The holes were first cleaned out and the edges trimmed. Stone, ranging from ¼ to 1 in. in diameter, was then placed in each hole and tamped, and to this there was applied about 1 gal. per square yard of a heavy asphalt binder. Chips were then spread over the surface and thoroughly tamped into the voids. Then a squeegee or seal coat was applied over the entire surface, including the stone patches. A recent inspection of this small strip of pavement showed that it was still in excellent condition, so that, without question, this method has already prolonged the life of that particular piece of pavement two years, and from indications will continue to prolong its life another two or three years. On account of the success of this experiment this method has been used for repairing sheet-asphalt pavement in two or three other small cities in various parts of the country, and up to the present time, reports have been very favorable. This method could be used very satisfactorily in small towns and villages, and would be much more economical than the use of brick or cement for repairing bituminous surfaces.

Rolling, Maintenance, and Engineering and Supervision in Illinois for Five Years.—These tables from the last report of the Illinois Highway Commission show costs for machinery owned and operated by the commission.

TABLE I.—COST FOR ROLLING ON 1,022,159 SQ. YDS. OF EXPERIMENTAL ROADS. 1908-1912.

Salary and expenses of roller operators	$27,593.00
Cost of maintenance of outfits	4,545.00
Freight on outfits	4,749.00
Total	$36,887.00
Cost per sq. yd. for rolling—	
Salary and expenses of operators	$0.0270
Maintenance of outfits	.0044
Freight on outfits	.0047
Total	$0.0361

TABLE II.—COST OF MAINTENANCE OF EXPERIMENTAL ROAD OUTFITS. 1908-1912.

Amount of road built, sq. yds.	1,022,159
Amount of bituminous macadam, sq. yds.	397,244
Cost of eleven macadam outfits	$37,579.00
Cost of six bituminous outfits	6,628.00
Cost of two concrete outfits	5,381.00
Total	$49,588.00
Cost of maintenance of macadam outfits	3,272.00
Cost of maintenance of bituminous outfits	1,273.00
Number of outfit seasons covered by maintenance charges on macadam outfits	52
Number of outfit seasons covered by maintenance charges on bituminous outfits	12
Cost per season per outfit for maintenance of macadam outfits	$ 61.80
Cost per season per outfit for maintenance of bituminous outfits	106.00
Cost per square yard for maintenance of macadam outfits	$0.003
Cost per square yard for maintenance of bituminous outfits	0.003

TABLE III. — COST OF ENGINEERING, SUPERVISION AND INSPECTION OF EXPERIMENTAL ROADS. 1908-1912.

Cost of work constructed	$650,183.00
Square yards of road constructed	1,055,501
Cost of 1,055,501 sq. yds. of road	$624,319.00
Total cost of engineering, superintendence and inspection, including office expenses for all road work	87,466.00
Less cost of engineering furnished on roads not built by the commission (25 miles at $50.00)	1,250.00
Less cost of supervision of non-experimental roads using penitentiary stone	1,335.00
Total expense not chargeable to experimental roads	2,585.00
Cost of engineering and supervision on experimental roads	84,881.00
Percentage of total cost of experimental roads expended for engineering superintendence and inspection, per cent	13
Cost per sq. yd. finished road for engineering, supervision and inspection	$0.059

Author Vol. 41, Jan-June/'14

Title Engineering-Contracting

35003

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